PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

COTTONWOOD DAM
WILSALL, MONTANA
PARK COUNTY
MT 21

prepared for

HONORABLE TED SCHWINDEN
GOVERNOR, STATE OF MONTANA

MONTANA DEPARTMENT OF NATURAL RESOURCES AND CONSERVATION
(OWNER-ADMINISTRATOR)

SHIELDS CANAL COMPANY
(OPERATOR)

prepared by

HKM ASSOCIATES
BILLINGS, MONTANA

FEBRUARY 1981

SEATTLE DISTRICT
UNITED STATES ARMY CORPS OF ENGINEERS

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TABLE OF CONTENTS

APPENDICES

EXECUTIVE SUMMARY

Personnel of HKM Associates, under a contract with the Montana Department of Natural Resources and Conservation (DNRC), and with representation from the DNRC (project owner/administrator), and the Shields Canal Company (project operator), inspected Cottonwood Dam on June 27, 1980. The inspection and evaluation were performed under the authority of Public Law 92-367. Cottonwood Dam is located on Cottonwood Creek, approximately 3.1 miles northwest of Wilsall, Park County, Montana. A contract for dam construction was awarded in July 1953, and the work was completed in December 1953.

FINDINGS AND EVALUATION

Cottonwood Reservoir stores runoff from a drainage of 34 square miles. The project was constructed to provide storage and regulation in support of irrigated agriculture. Incidental benefits related to the storage project are recreation, sediment trap, and floodwater detention. Storage capacity to the existing spillway crest elevation is 1900 acre feet (AF). Total storage capacity to first overtopping dam crest elevation is 3670 AF. The dam has a hydraulic height of 39 feet and a structural height of 51 feet. On the basis of criteria in the U.S. Army Corps of Engineers Recommended Guidelines for Safety Inspection of Dams (Ref. 1), the project is intermediate in size. There is a high road embankment (State Highway 89) in the flood plain immediately downstream of the dam. The road embankment is an earthen structure, slightly lower in height than Cottonwood Dam, and contains a 96-inch circular pipe. It is assumed that the road embankment will overtop and fail following a breach of Cottonwood Dam, and therefore will not aid in flood detention and attenuation. Failure of Cottonwood Dam, and subsequent failure of the highway embankment, could endanger many lives and cause extensive property damage. In particular, there is a state highway, private and county roads, miscellaneous power, phone, water, and sewer services, irrigation distribution systems, and three (3) residences that would be impacted by a breach of Cottonwood Dam. The downstream hazard potential is therefore high (Category 1). However, no dam breach analysis or routing of a dam breach flood was made for the downstream area. The conclusions on probable damage are based on a brief field inspection and engineering judgment.

The guidelines recommend that the discharge and/or storage capacity of an intermediate-size, high downstream hazard potential dam be capable of safely handling the Probable Maximum Flood (PMF). The PMF is the flood expected from the most severe combination of meteorologic and hydrologic conditions that are reasonably possible in the region. Routing of the estimated PMF

developed for the safety evaluation of Cottonwood Dam showed that the project has the capacity of controlling a flood having hydrograph ordinates approximately equal to 12 percent of the PMF hydrograph ordinates. However, because of the poor condition of the spillway (for details, see page 73) which may not be able to sustain high flows without failure, the project "capability" is considered to be less than its "capacity". A detailed analysis was not performed to determine the maximum "capability" because of the informational base available and the scope of a Phase 1 Investigation. In any case, the embankment materials for Cottonwood Dam are classified as being moderately erodible and can be expected to remain stable only for a short period of time under an overtopping condition.

Some seepage areas were identified during the field investigation. Seepage appears to be passing through the abutments or under the dam. However, specific locations of the seepage paths and the phreatic surface through the dam area are unknown due to a limited informational base and the fact that field measurements could not be made. Seepage was also identified in the area of the spillway chute.

The results of the surficial examination of Cottonwood Dam generally indicates that the embankment structure is stable and in good condition. A more complete and definitive evaluation of embankment stability could not be made because of insufficient information. There has been a loss of some riprap on the upstream face of the dam and in the spillway stilling basin. In both areas, the loss of riprap has exposed erodible soils.

A comparison of report findings with inspection guidelines shows Cottonwood Dam to have insufficient storage and/or discharge capacity to safely handle the recommended spillway design flood (SDF), which is the full PMF. Since the project cannot handle one-half the PMF and the spillway is in poor condition, it is considered seriously inadequate. According to inspection guidelines, the project is considered unsafe, nonemergency until recommended actions are complete.

RECOMMENDATIONS

Immediately develop, implement and test a downstream warning plan. Special immediate attention and rehabilitation efforts must be directed to the spillway because of its deteriorating condition and inadequate capacity. Recommended temporary improvements to the spillway include removing the flashboards, repairing the concrete floor to minimize water travel beneath the floor, and repairing or replacing the catwalk. It is recommended that large discharges through the spillway be avoided until it is repaired or replaced. In general, this can be accomplished by lowering the reservoir pool as much as reasonably possible in the winter to maximize the availability of flood storage. Suggested

improvements/repairs to the stilling basin include removal of the rock from between the baffle blocks, rearrangement/replacement of rock riprap downstream of the blocks and near the basin wing walls, and repair of the concrete cracking of the blocks and walls. Inspect the outlet works from the valve upstream to the intake structure, and repair as necessary. Recommended repairs to the outlet works from the operating valve downstream to the stilling basin include: the repair of the gap in the 36-inch corrugated steel pipe (CSP) immediately downstream of the valve; gate seat repair; cleaning and reapplication of a protective coating to the interior of the CSP; replacement of deteriorated portions of the paved invert along the pipe flowline; replacement of the joint filler material at the pipe joints; application of a protective material to the contact area between the cast iron vent pipe and the 36-inch CSP to stop localized corrosion; and application of a filler or grout material along the stilling basin wall where the outlet works pipe daylights. Replacement/rearrangement of rock riprap is suggested along the upstream face of the embankment where the rock has moved downslope. Minor repair is required along the dam and dike crest where small depressions have developed. Install piezometers in the embankment, abutments, and foundation, and establish a regular monitoring program to evaluate piezometric conditions.

Conduct more detailed hydrologic and hydraulic routing studies to better determine the downstream hazard and required spillway capacity and to evaluate embankment stability. Modify the project as studies indicate. Conduct periodic inspections of the project at 5-year intervals by engineers experienced in dam design and construction. Develop and implement a periodic maintenance plan for the dam and appurtenant structures. The project operator should contact the Montana DNRC, Dam Safety Section, prior to performing any remedial construction to insure compliance with all pertinent laws and regulations.

Mike Keene

Michael D. Keene
Professional Engineer

MONTANA
MICHAEL D.
KEENE
4197 E
REGISTERED
PROFESSIONAL ENGINEER

PERTINENT DATA SUMMARY

1. General

Federal ID Number	MT 21
Owner and Project Administrator	Montana Department of Natural Resources and Conservation
Operator	Shields Canal Company
Purpose	Irrigation
Location	Sections 1, 2, 11, and 12, T3N, R8E, MPM; 3.1 miles northwest of Wilsall, MT
County, State	Park County, Montana
Watershed	Cottonwood Creek (tributary to Potter Creek)
Hazard Potential	Category 1 (High)
Size Classification	Intermediate

2. Reservoir

Surface Area at Spillway Crest Elevation (with Flashboards) 82.5 feet	235 acres
Dead Storage to Elevation 52.0 feet (Invert of Outlet Works Inlet)	0 acre-feet
Active Storage to Spillway Crest Elevation (with Flashboards) 82.5 feet	1900 acre-feet
Total Storage to First Overtopping Dam Crest Elevation 88.74 feet	3670 acre-feet
Total Storage to Dam Crest Design Elevation 89 feet	3800 acre-feet

PERTINENT DATA SUMMARY
(Continued)

Total Drainage Area	34 square miles
Reservoir Water Surface Elevation on Day of the Inspection	82.5 feet

3. Spillway

Crest Elevation with Flashboards	82.5 feet
Crest Elevation without Flashboards	80.0 feet
Type	Designed as an uncontrolled ogee crest and concrete chute. Control now provided by permanent timber flashboards.
Crest Width	20 feet
Chute Width	20 feet
Spillway Capacity with Flashboards (to First Overtopping Dam Crest Elevation)	1030 cubic feet per second

4. Outlet Works

Gate	36 inch diameter slidegate valve
Control	Manual operator
Conduit	197 feet of 36 inch diameter corrugated steel pipe, 10 gauge, double bituminous coated with paved invert
Capacity To Spillway Crest	105 cubic feet per second

To First Overtopping Dam Crest Elevation	112 cubic feet per second

5. Dam

Type	Zoned earth fill
Structural Height	51 feet
Hydraulic Height	39 feet
Design Crest Control Elevation	89.0 feet
Existing First Overtopping Dam Crest Elevation	88.74 feet
Total Crest Length	1435 feet
Dam	580 feet
Dike	825 feet
Spillway	30 feet (on diagonal)
Design Dam Crest Width	20.0 feet
Design Dike Crest Width	10.0 feet
Existing Dam Crest Width	Variable 20-26 feet
Existing Dike Crest Width	Variable 10-18 feet
Upstream Dam Slope	1V on 3H
Upstream Dike Slope	1V on 4H
Downstream Dam Slope	1V on 2H
Downstream Dike Slope	1V on 2H

Note: All project elevations are relative to a local survey datum.

CHAPTER 1
BACKGROUND

1.1 INTRODUCTION

1.1.1 Authority

This report summarizes the Phase I inspection and evaluation of Cottonwood Dam. The project is owned and administered by the Montana Department of Natural Resources and Conservation (DNRC), and operated and maintained by the Shields Canal Company.

The National Dam Inspection Act, Public Law 92-367 dated August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to conduct safety inspections of non-federal dams throughout the United States. Pursuant to that authority, the Chief of Engineers issued "Recommended Guidelines for Safety Inspection of Dams" in Appendix D, Volume 1 of the U.S. Army Corps of Engineers' Report to the United States Congress on "National Program of Inspection of Dams" in May 1975.

The recommended guidelines were prepared with the help of engineers and scientists highly experienced in dam safety from many federal and state agencies, professional engineering organizations and private engineering consulting firms. Consequently, the evaluation criteria presented in the guidelines represent the comprehensive concensus of the engineering community.

Where necessary the guidelines recommend a two-phased study procedure for investigating and evaluating existing dam conditions so deficiencies and hazardous conditions can be readily identified and corrected. The Phase I study is:

(1) a limited investigation to assess the general safety condition of the dam

(2) based upon an evaluation of the available data and a visual inspection

(3) performed to determine if any needed emergency measures and/or if additional studies, investigations and analyses are necessary or warranted

(4) not intended to include extensive explorations, analysis or to provide detailed alternative correction recommendations.

The Phase II investigation includes all additional studies necessary to evaluate the safety of the dam. Included in Phase II, as required, should be additional visual inspections, measurements, foundation exploration and testing, material testing, hydraulic and hydrologic analyses and structural stability analyses.

The authority for the Corps of Engineers to participate in the inspection of nonfederally owned dams is limited to Phase I investigations with the exception of situations of extreme emergency. In these cases the Corps may proceed with Phase II studies but only to the extent needed to answer serious questions relating to dam safety that cannot be answered otherwise. The two phases of investigations outlined above are intended only to evaluate project safety and do not encompass in scope the engineering required to perform design or corrective modification work. Recommendations contained in this report may be for either Phase II safety analyses or detailed design study for corrective action.

The responsibility for implementation of these Phase I recommendations rests with the State of Montana, Department of Natural Resources and Conservation. The operator is urged to contact the Montana DNRC prior to taking any action on report recommendations. It should be noted that nothing contained in the National Dam Inspection Act, and no action or failure to act under this Act, shall be construed (1) to create liability in the United States or its officers or employees for the recovery of damage caused by such action or failure to act or (2) to relieve an owner or operator of a dam of the legal duties, obligations, or liabilities incident to the ownership or operation of the dam.

The investigation process allows for report review by: the Montana DNRC (project owner/administrator) and the Shields Canal Company (project operator). Review comments are considered before final publication of the Phase I Inspection Report. Their written comments are enclosed in Appendix E.

1.1.2 Purpose and Inspection

The findings and recommendations in this report were based on visual inspection of the project, minimal field survey measurements, and review of available design and operational data. The purpose of the inspection is to make a general assessment as to the structural integrity and operational adequacy of the dam embankment and its appurtenant structures. Inspection procedures and criteria were those established by the Recommended Guidelines for Safety Inspection of Dams (Ref. 1).

The visual inspection of Cottonwood Dam was made on June 27, 1980. HKM Associates personnel who attended the field inspection and contributed to this report were:

Dan Dyer, Geotechnical Engineer
Gary Elwell, Hydraulics/Hydrology
Mike Keene, Hydraulics/Hydrology, Team Leader

Other HKM personnel contributing to the report but not attending the field inspection were:

Dale R. Cunnington, Structural Engineer
William Hansen, Hydraulics/Hydrology
Dan Nebel, Geology

Other personnel present during the June 27, 1980 inspection included:

Glen McDonald, Supervisor, Montana DNRC, Dam Safety Section
Larry Tegg, Dam Safety Engineer, Montana DNRC, Dam Safety Section
Carl Arthun, President, Shields Canal Company
Len Arthun, Shields Canal Company
Bud O'Halloran, Shields Canal Company

1.2 DESCRIPTION

1.2.1 General

Cottonwood Dam is a zoned earth fill dam located in the SW 1/4 of Section 1, T3N, R8E, M.P.M., Park County, Montana (Appendix A and Ref. 2, 3, 4, 5). The dam is composed of a main dam embankment and a dike forming a single, continuous structure.

The dam and reservoir form an irrigation facility within the Missouri River Basin by containing the water of Cottonwood Creek and tributaries. Cottonwood water is returned directly to Cottonwood Creek. Cottonwood Creek travels approximately 400 feet before reaching a major highway embankment, and then another 200 feet before joining Potter Creek (Appendix A and Ref. 2). Potter Creek continues another 2.6 miles until joining Flathead Creek. The nearest downstream community is Wilsall, Montana, which is located on the west bank of Flathead Creek approximately 3.1 miles southeast of the dam. In terms of a stream channel distance, Wilsall is located 4.5 miles downstream of the dam. There are several homes located in the floodplain of Flathead Creek.

The Shields Canal Company operates and maintains the project. The State of Montana administers the project.

Cottonwood Dam has a hydraulic height of 39 feet and impounds 3670 AF at the first overtopping dam crest elevation (elevation 88.74 feet). Based on a visual reconnaissance and engineering judgment, three residences, as well as roads, bridges, and agricultural land will be affected by a sudden breach of the dam. Of particular importance would be the highway embankment immediately downstream of the dam. It is assumed the highway embankment will breach in the event of a Cottonwood Dam breach. On the basis of this information and in accordance with the Recommended Guidelines (Ref. 1), the project is classified intermediate in size and the downstream hazard potential is high (Category 1).

Cottonwood Dam was constructed to provide storage and regulation in support of irrigation practices. Incidental benefits are provided for floodwater detention, sediment accumulation, and recreation. Active storage to the normal pool level, or the spillway crest with flashboards (elevation 82.5 feet), is 1900 AF (Ref. 5). An additional 1770 AF are available for flood surcharge storage between the spillway crest with flashboards and the first overtopping dam crest elevation.

Cottonwood Dam has a total upstream contributory drainage area of 34 square miles. The Cottonwood watershed is characterized by both foothill and prairie drainage. Elevations in the basin range from 7325 feet NGVD to about 5100 feet NGVD at the reservoir (Ref. 6).1/ The reservoir is located in open, generally flat terrain (Photo 1 of Appendix B).

1.2.2 Regional Geology

Cottonwood Dam is situated near the north end of the Shields River valley in central Montana. The topography is comprised of low rounded hills and shallow alluvial valleys. Structurally, the dam lies near the north end of the axis of the Crazy Mountain syncline and is bordered by the Crazy Mountain uplift to the east, the Bridger Mountain uplift to the west, and the Smith River-Shields River divide to the north. The strata in the area shows a uniform northwesterly dip of 10 to 15 degrees with the Livingston Formation forming the surface of the area. Numerous small faults are present in the immediate area, however, there is no evidence of recent activity (Ref. 7, 8).

1/ Basin elevations in feet NGVD were obtained from the White Sulphur Springs AMS map. All other elevations contained in this report are local datum.

1.2.3 Seismicity

Cottonwood Dam is in a moderately active to active seismic zone with the majority of the region's seismic events occurring in the Southwestern Montana-Yellowstone Park area. Since 1925, Montana has experienced five shocks that reached intensity VIII or greater (Modified Mercalli Scale). The closest epicenter occurred in the Three Forks, Montana area which is approximately 38 miles west of the damsite. Numerous other shocks of intensity IV or greater have been reported within a 100-mile radius of the site (as of January 1974). The site is located in zone 3 of the Seismic Zone Map of Contiguous States, and it can be assumed that a major earthquake may occur within the life of the structure (Ref. 1, 9). Although the Zone Map is based on a known distribution of damaging earthquakes, it does not necessarily reflect accurate or adequate seismic design parameters for this site.

1.2.4 Site Geology

Eugene S. Perry of the Montana School of Mines completed a geologic reconnaissance of the dam and reservoir site in June 1949. In addition, six test pits were completed along the valley portion and left abutment of the dam during the field season of 1951 and at least nine holes were drilled in the dam and dike area for installation of grout pipes in 1954 and 1955. This section of the report is developed from these sources plus personal knowledge of the area (Ref. 7).

The reservoir basin is formed in alternating beds of andesitic shaly sandstone and dense shale of the Livingston Formation. The alternating sandstone and shale layers vary in thickness from 1 foot to 20 feet. The sandstone and shale both readily undergo decomposition when exposed to the air with the andesitic sandstone grains decomposing to clay. The valley section consists of shallow alluvial and colluvial deposits of sand, silt and clay loam, extending to a maximum depth of 12 feet, over gravel and weathered shale and sandstone.

The outlet works and stilling basin are located in alluvium which has been protected by angular riprap. During the field inspection it was noted that the riprap is slumping with erosion occurring along the stilling basin perimeter, particularly on the right side.

1.2.5 Sedimentation

The catchment area above Cottonwood Reservoir consists primarily of grass-covered rangeland and foothills. Water in the reservoir pool at the time of the survey was very murky, but this was probably due to the high winds rather than an indicator of a high

sediment yield watershed. Sediment is building up at the approach to the spillway as stated in the project file (Ref. 7). Further study would better define the sediment yield from the watershed and if sediment deposition in the pool is becoming a problem.

1.2.6 Design and Construction History

Cottonwood Dam was designed and constructed by the Montana State Water Conservation Board (SWCB). The eventual project operator, Shields Canal Company, had been incorporated for about five years prior to the construction of the storage project. A water marketing contract was drafted between the SWCB and the Shields Canal Company on June 30, 1953. The construction contract was awarded to Stanley H. Arkwright, Inc., Billings, Montana on July 7, 1953, and the project was completed in December 1953.

It appears that six (6) test pits were constructed on the left (east) side of Cottonwood Creek during the early phases of the design. Soil tests were performed by the SWCB on selected samples taken from the borrow areas and test holes. The tests included: moisture-density relationships, grain-size distribution, and permeability. The results of these tests are available in the project file (Ref. 7).

It was not definitive as to whether any of the soil tests for embankment design were performed during construction. Gradation tests were performed on samples during construction, but it appeared as through these tests related specifically to concrete mix design. Other quality control work performed during construction was concrete compressive strength testing. Records indicate that an inspector, Mr. Lester Martin, was on-site during at least part of the construction (Ref. 7).

Since completion of the original construction work, repair and/or improvement work has apparently consisted of a grout hole drilling program and the installation of flashboards at the spillway inlet. During the period spanning from December 1954 to January 1955, Al Reidalbach (apparently an independent driller) supposedly drilled at least 9 grout holes. Drill logs were found in the project file for 9 holes. It appears that a majority of the holes were drilled along the dam crest. No details were provided in the file explaining the purpose of the drilling and the success of the grouting program if completed (Ref. 7).

About three years after construction, timber flashboards were installed in the spillway. The purpose of the installation was to develop more storage to better compliment and support the downstream irrigation practice. Design and installation was performed by Mr. Carl Arthun, a member of the Shields Canal Company. A repair was made to the flashboards approximately four years ago when the braces were replaced due to rotting.

CHAPTER 2
INSPECTION AND RECORDS EVALUATION

2.1 HYDRAULICS AND STRUCTURES

2.1.1 Spillway

The spillway for Cottonwood Dam is located on the right (south) side. The spillway was originally designed as a concrete lined chute having a gravel approach section and a concrete ogee crest. Approximately three years after construction wooden flashboards were permanently installed. The approach channel has a length of no more than 10 to 15 feet. The ogee crest was set at elevation 80.0 feet. The wooden flashboards raise the crest elevation 2.5 feet to 82.5 feet. A wooden foot bridge spans the spillway. Spillway width at the ogee and in the chute is 20 feet. Energy dissipation is provided by a concrete stilling basin containing baffle blocks (Sheet 4 of Exhibit C1, Appendix C). There is a 34.5-foot drop from the existing spillway crest to the design stilling basin invert. Spillway flows pass from the stilling basin into Cottonwood Creek. Pertinent construction plans are provided on Sheets 1 and 4 of Exhibit C1.

Original hydraulic design information for spillway operation is not available. Therefore, it was necessary to develop new hydraulic rating information. This new rating was developed using the weir head-discharge relationship with a discharge coefficient of 3.3. This discharge coefficient was chosen because the wooden flashboards cause the spillway control to function as a sharp-crested weir. Energy losses in the approach channel were assumed to be negligible due to the very short length even though the spillway entrance is quite abrupt. Spillway capacity to the first overtopping dam crest elevation (88.74 feet, local datum) is estimated to be 1030 cfs. Spillway capacity could be about doubled with removal of the flashboards. The chute walls are of adequate height to contain all spillway flows. Spillway rating data is provided in Exhibits D2 and D3 of Appendix D.

Inspection reports indicate a sediment buildup at the spillway entrance, but this does not appear to present a serious operational problem at this time (Ref. 7). The alignment of the spillway is poor relative to the location of the reservoir pool, which can result in erosion at the spillway entrance and minor reduction of spillway capacity.

The permanent wooden flashboards appear to be in good condition. The timber members that support the wooden flashboards could collect large debris and, therefore, form an obstruction for spillway flows. A log boom is not used. The timber catwalk

across the spillway crest is in very bad condition with the handrails missing (Photo 2 of Appendix B). The condition of the catwalk has been mentioned in previous inspection reports (Ref. 7).

The spillway chute shows a high degree of deterioration. The entire chute floor is very rough from erosion. Large cracks have developed in the concrete floor. Near the top of the spillway chute, concrete spalling has occurred along longitudinal construction joints in the spillway floor leaving the metal water stops exposed which has allowed water to seep under the spillway floor (Photo 3 of Appendix B). Approximately half way up the chute there are a number of cracks which appear to be the result of settlement and/or uplift. Water is escaping under pressure from underneath the floor slab at the bottom of the chute (Photo 4 of Appendix B). At this location the center floor slab has been displaced upward approximately 3 inches. Despite the presence of water underneath the floor slab, none of the weep holes in the floor were functioning at the time of the survey. The weep holes were filled with small stones and debris. It was possible to unplug a few by cleaning them out. The chute walls are vertical and appear to be stable.

The stilling basin had numerous deficiencies. There is seepage coming from a large crack formed in the north stilling basin wall immediately upstream of the outlet works pipe. Excess separation was observed in some of the concrete construction joints. Differential settlement of the stilling basin floor was evident during pumping/draining of the basin for inspection purposes. As the water level in the stilling basin lowered, the baffle blocks were exposed at different elevations, which is contrary to the design drawings. Considerable erosion has taken place around the stilling basin wing walls, with the most severe erosion occurring at the right (south) wing wall. Erosion is probably concentrated on the right side because the outlet works pipe exits in this direction. The end of the right wing wall is completely exposed. There is minimal riprap protection for the wing walls. A large amount of silty material has deposited in the stilling basin floor, and some rock riprap has deposited in the basin immediately downstream of the baffle blocks.

2.1.2 Outlet Works

The outlet works for Cottonwood Dam is located about 130 feet from the right abutment contact and aligned perpendicular to the dam crest. The outlet facility consists of: a concrete inlet structure; a 36-inch diameter conduit; a wet-well chamber (a control tower which contains the operating gate); cutoff collars; an air vent pipe; a concrete outlet structure and stilling basin (common with the spillway); and a return channel. Construction plans are provided on Sheets 1, 2, 3 of Exhibit C1.

The intake was designed as a flared inlet with a metal trashrack. Flowline elevation for the intake was set at 52.0 feet (Sheet 2 of Exhibit Cl). The intake structure could not be inspected due to the height of the reservoir pool at the time of the inspection.

The outlet conduit is a 36-inch diameter 10 gauge, corrugated steel pipe (CSP). Pipe sections were connected with watertight band couplers. It received a factory applied double bituminous coating and was furnished with a paved invert. Total conduit length from intake structure to outlet structure is about 200 feet. The conduit was inspected only from the operating gate downstream to the outlet during the June 1980 inspection. Much of the paved invert of the pipe has been removed by erosion. The asphaltic interior coating of the pipe is flaking or completely gone in places, especially at the downstream end of the pipe. Much of the joint material at the watertight connections is deteriorating or has been removed, particularly along the flowline of the pipe. A gap has developed along the pipe invert immediately downstream of the gate valve. A portion of the CSP invert has been bent upward 1 to 2 inches. However, this gap does not expose natural soil or aggregate, probably because of a coupling band. There is a slight amount of corrosion occurring at the contact of the 4-inch cast iron vent pipe and the CSP pipe crown. There are some slight horizontal and vertical misalignments of the outlet pipe, but are not significant enough to consider them a deficiency or potentially hazardous condition. The entire length of the outlet pipe has approximately the same gradient. Three reinforced concrete cutoff collars are placed at 25-foot spacings along the outlet pipe upstream of the wet-well chamber. Two cutoff collars, spaced 24 feet apart, are located downstream of the wet-well chamber.

The control tower is located 12.6 feet upstream of Cottonwood Dam centerline. The tower contains a 36-inch diameter slidegate which is mounted on the east (downstream) wall. The gate operating device is located on the top floor level of the control tower, which is approximately coincident with the dam crest elevation. The gate house was never constructed, hence the gate operating device is exposed (Photo 8 of Appendix B). At the time of the inspection the operating gate was fully closed. A complete seat of the slidegate valve was not possible. Approximately 10 to 15 gallons per minute (gpm) were seeping through the downstream northwest corner of the gate. There are small seeps in the northeast corner, but they are very minor in comparison to the major seeping. After inspection of the pipe and gate, the gate was opened a short distance to verify operational capabilities. There was no physical or operational evidence that would indicate that the gate could not be fully opened.

The outlet pipe and spillway share the same stilling basin. See Section 2.1.1 for inspection comments. It was also noted that there is a gap in the concrete around the outlet pipe where the pipe daylights at the stilling basin (Photo 5 of Appendix B).

Outlet works capacity, with the reservoir at the spillway crest with flashboards (elevation 82.5 feet), is estimated to be approximately 105 cfs, and 112 cfs to the first overtopping dam crest elevation (88.74 feet). Estimates are based on the assumptions that the operating gate is in the fully open position, full conduit flow exists throughout the entire length, and that a representative value for Manning's "n" is 0.025. Final outlet works rating data is summarized in Exhibits D2 and D3 of Appendix D.

2.1.3 Freeboard

Flood routing (Section 2.2.3) indicates the dam overtops during the guidelines' (Ref. 1) recommended spillway design flood (which is the full PMF), and therefore, no freeboard exists for such conditions. The vertical distance from the spillway crest with flashboards (elevation 82.5 feet) to the first overtopping dam crest elevation (88.74 feet) is 6.24 feet. The project operator indicated that a probable historic highwater level for Cottonwood Dam occurred in the spring of 1979. It was estimated that the flow depth over the spillway crest (with flashboards) as about 6 inches. Highwater marks on the left spillway wing wall substantiate this probable historic highwater level. The vertical distance between the reservoir pool and the first overtopping at the time of the June 1980 field inspection was 6.24 feet.

Cottonwood Reservoir is basically oriented in a southeast-northwest direction, with the dam on the southeast side. The prevailing wind for this region is generally identified as being westerly (Ref. 10, 11). The reservoir location and orientation can be observed in Appendix A. The effective fetch length is calculated to be 0.9 mile. Hence, a minimum freeboard allowance should be about 3 feet (Ref. 12). This 3-foot allowance does not apply to an event as severe as the PMF where the design objective would be to provide only enough freeboard to maintain structural integrity of the dam. Although the dam will overtop during the PMF and lesser floods, the vertical distance between normal pool and dam crest is sufficient to prevent overtopping by wind-generated waves.

2.2 HYDROLOGY

2.2.1 Physiography and Climatology

The 34-square mile catchment area above Cottonwood Reservoir is basically rectangular in shape, with the length dimension being considerably greater than width (Appendix A). In particular, the drainage area is approximately 11 miles long and only 4 miles wide. The topography includes rolling prairies and foothills. The immediate vicinity of the reservoir can primarily be characterized as prairie drainage. The watershed rises from the reservoir in a northwesterly direction by gradual slopes and bench lands to the foothills of the Bridger Range.

Soils along the stream bottoms are generally alluvial soils having characteristics of the low stream terraces. Bench-land soils consist of Big Timber-Castner soils (Ref. 13). The regional climate is classified as distinctly continental, and characterized by abundant sunshine, low relative humidity, light rainfall, and wide daily and seasonal variations in temperature. However, the regional climate does not have the extreme variable pattern common to the more mountainous western sections in Montana. In general, the valleys are relatively dry during the colder months and wet during the late spring and early summer. The wettest part of the year in the mountains is generally from midwinter to early spring. It is not uncommon for the region to experience winter warming spells with associated thawing temperatures. Precipitation in the Cottonwood watershed ranges from about 15.5 inches at Wilsall (3.1 miles southeast of the dam) to amounts in excess of 30 inches in the foothills of the Bridger Range. The average annual temperature at Wilsall is approximately 42 degrees Fahrenheit. Winters are typically cold, with January being the coldest month. The monthly average temperature for January at Wilsall is about 25 degrees Fahrenheit. During the summer, July is typically the warmest month with an average temperature of about 64 degrees Fahrenheit (Ref. 10, 11).

No streamflow measurements on Cottonwood Creek or pool level records on Cottonwood Reservoir are available.

2.2.2 Estimated Probable Maximum Flood (PMF)

The probable maximum precipitation (PMP) and the estimated probable maximum flood were developed for the Cottonwood drainage basin. The ratio of the reservoir area to the non-reservoir area is less than 1 percent; therefore, the two areas were not separated for the purpose of this analysis.

The PMF is the flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the study region.

Cottonwood Reservoir is located west of the 105th meridian and east of the Continental Divide, hence PMP values were calculated using procedures associated with the National Weather Service's Interim Method (Ref. 14). The Interim Method provides PMP values for durations of 6 to 72 hours, with PMP values for 1 to 5 hours obtained by multiplying the 6-hour PMP value by percentages supplied by the Corps of Engineers, Seattle District. The PMP 6-hour, 12-hour, 24-hour, 48-hour, and 72-hour values were 9.4 inches, 12.2 inches, 14.7 inches, 17.6 inches and 19.7 inches, respectively. The 6-hour PMP value of 9.4 inches was multiplied by 67 percent, 78 percent, 85 percent, 91 percent, and 95 percent to obtain PMP values of 6.3 inches, 7.3 inches, 8.0 inches, 8.6 inches and 8.9 inches for durations of 1-hour, 2-hours, 3-hours, 4-hours, and 5-hours, respectively.

The 6-hour increments for the total 72-hour storm were arranged in a critical distribution using criteria presented in the National Weather Service's Hydrometeorological Report No. 43 (Ref. 15). In particular, the 6-hour rainfall increments were arranged according to pattern "e". Further subdivision of the calculated rainfall increments was required to provide compatibility with the duration of the unit hydrograph. A 30-minute unit hydrograph was chosen for Cottonwood Reservoir using criteria presented in the SCS Hydrology Handbook (Ref. 16). The unit hydrograph was developed for the Cottonwood basin using the U.S. Army Corps of Engineers' computer program HEC-1 and the SCS method (Ref. 16, 17). The PMP storm was plotted in the form of a depth-duration curve for convenience in selecting incremental rainfall values. The 30-minute PMP values, within the time base of the unit hydrograph, were ordered according to the reverse pattern of the unit hydrograph ordinates.

Rainfall losses were assumed equal to the minimum soil retention rate of 0.15 inches/hour for type B soils (soils having a moderate infiltration rate) due to an assumed saturated ground condition (Ref. 10).

The runoff condition, or PMF, resulting from a PMP storm was estimated using the PMP values and the unit hydrograph approach. The resultant PMF has a peak flow of 57,440 cfs and a 72-hour volume of 20,210 AF.

2.2.3 Flood Routing

The PMF resulting from the PMP rainfall/runoff event was routed through Cottonwood Reservoir using the computer program HEC-1 (Ref. 17). Runoff from an antecedent storm was not specifically considered; however, it appears reasonable to assume the initial reservoir level prior to routing the PMF is at the spillway crest with flashboards (elevation 82.5 feet). The support rationale for this assumption is that Cottonwood Reservoir characteristically has a small reservoir pool and is not capable of totally absorbing rather severe flood events. Drawdown of the reservoir pool can be accomplished with the 36-inch outlet conduit. However, it is not likely that the project would be operated in this manner unless it was an emergency measure. The water users would prefer to store as much water as possible in preparation for the irrigation season. In summary, the "sense of emergency" is not likely to be realized by the project operators to the extent storage water would be sacrificed in anticipation of the PMF event.

Reservoir area-capacity-elevation data was obtained from a DNRC publication (Ref. 5), and is provided in Exhibit D1 of Appendix D. Cottonwood discharge rating data is shown in Exhibits D2 and D3 of Appendix D. Flow through the outlet works was not included in the flood routing since the slidegate would probably be closed to store water for the irrigation season.

For the purposes of flood routing and according to Seattle District, Corps of Engineers, Phase I investigation criteria, the dam crest elevation is the minimum elevation to which the reservoir must rise before overtopping the dam. This criteria assumes overtopping and failure of embankment-type dams to be coincidental. Based upon the embankment profile survey dated June 27, 1980 (Exhibit C2 of Appendix C), the existing low-point dam crest elevation is 88.74 feet.

Flood routing showed that the dam would first overtop during the PMF when approximately 11 percent of the total PMF volume enters the reservoir. Routings were made of lesser hypothetical floods than the PMF to determine the magnitude of floods the dam can contain. The hypothetical hydrographs are obtained by applying percentages to the PMF ordinates. A flood with a hydrograph having ordinates corresponding to 12 percent PMF ordinates is just controlled by the project. Larger floods would overtop the dam.

2.3 GEOTECHNICAL EVALUATION

The geotechnical evaluation of Cottonwood Reservoir included a field investigation and a search and review of project data. The field inspection consisted of photo documentation, a dam crest profile survey, slope stability observations of the dam embankment, seepage observations, and measurements of the slope angles. Inspection photos are included in Appendix B, some of the construction plans are included in Exhibit C1 of Appendix C, and the crest profile survey is shown in Exhibit C2. Results of the slope angle measurements are provided in Exhibit C3.

2.3.1 Dam Embankment

Cottonwood Reservoir is a compacted, zoned earth fill structure which was completed in December 1953. The dam has an estimated maximum structural height above the deepest point on the foundation surface of 51 feet and a total crest length of 1435 feet. The main dam embankment is about 610 feet between dam abutments, with an earth fill dike extended 825 feet beyond the left abutment. Included in the main embankment section is a 20-foot-wide concrete spillway (30 feet diagonal along dam center line). The dam and dike crest widths were designed to be 20 and 10 feet, respectively. In addition, crest width of about 7.5 and 4 feet were added to the dam and dike, respectively by the placement of riprap on the upstream face. Field measurements indicate variable widths of between 20 and 26 feet on the main dam (Photo 6 of Appendix 13) and 10 to 18 feet on the dike section. The narrowing of the crest on the main dam from 27.5 feet is primarily the result of the riprap moving down the upstream slope. The actual dike crest is wider than shown on the construction drawings. The reason for this discrepancy is unknown. A thin, fine gravel course has been placed on the crest of the main dam.

The construction plans (See Sheet 1 of Exhibit C1) indicate the upstream face of the main dam has a slope of 1V on 3H and for the dike, a slope of 1V on 4H. Slope measurements in the field verify these slopes. The upstream face was designed with a 30-inch to 18-inch cover of loose rock riprap varying in thickness from crest to toe, respectively, on the main dam. There is a 12-inch sand and gravel bedding blanket underlying the rock riprap. On the dike section, there is a 12-inch riprap blanket of sand and gravel. The present condition of the riprap will be discussed in Section 2.3.4. There is a moderate cover of grasses, weeds, and sagebrush on the upstream slope above the water surface (Photo 6 of Appendix B). Debris on the upstream face is limited to very small material.

The downstream face of the dam and dike has a slope of 1V on 2H. There is a moderate cover of natural grasses, weeds, and sagebrush on the downstream slope (Photo 7 of Appendix B).

Details provided on Sheet 5, Exhibit C1 indicate the main dam embankment cross section consists of an impervious fill (core), random earth and gravel (shell), and riprap with a sand and gravel blanket (bedding). A detailed description of these materials and zones is not available in the project design file. These sections are not identified by zone numbers.

The impervious zone is located almost entirely upstream of the embankment centerline and includes about 60 percent of the cross section. Plans indicate that a cutoff trench is located at the base of the impervious zone. The drawings also indicate the cutoff trench is up to 16 feet deep and extends into the underlying bedrock about 4 feet (Sheet 5 of Exhibit C1). There is no cutoff trench under the dike section.

The earth and gravel zones form the upstream (below the riprap and riprap bedding) and downstream embankment shells. Based on field observations, it appears a thin layer of topsoil was placed over the earth and gravel zones on the downstream face and above the water surface on the upstream face (Photo 8 of Appendix B).

There are small berms at both the upstream and downstream toes of the dam. According to the drawings, the upstream berm is waste material from the foundation stripping obtained during construction; the downstream berm is composed of earth and gravel materials and appears to be covered with some topsoil. The top of the downstream berm is sloped away from the toe rather than level as shown on the drawing (Sheet 5 of Exhibit C1).

2.3.2 Foundation Conditions and Seepage Control

Six (6) test pit logs were available for review. All of these test pits were made in the left abutment and in the valley floor on the left side of Cottonwood Creek. No test holes were made in the right abutment area during the initial investigation. In general, these logs indicate that there are alluvial deposits of sand, gravel, and clay from 4 to near 12 feet thick on the left abutment and in the valley floor.

Laboratory tests were performed by the Montana State Water Conservation Board on selected samples taken from the borrow areas and test holes. These tests included: moisture-density relationships, grain-size distribution, and permeability tests. The results of these tests are available in the design file (Ref. 7). It is not known with certainty what zones of material the tests represent.

In December of 1954 and January 1955, approximately one year after the dam was constructed, at least 9 holes were drilled in the dam and dike area for the installation of grout pipes. No details were provided in the file explaining the purpose of the drilling and the success of the grouting program if completed (Ref. 7). The pipes were evidently left in the hole for the purpose of monitoring the phreatic surface in the embankment. A review of these drill logs indicate the holes were drilled to depths of from 30 to 60 feet below the crest elevation of 89.0 feet. Only two of these holes could be located in the field. These locations are approximated on Exhibit C3 of Appendix C. The project operator indicated that it is likely that the other holes have been covered as a result of traffic and vegetation. The two pipes found had been filled with debris and water table measurements could not be found. A history of water table measurements is not available.

Based on a review of the test pit logs, it appears that the main section of the existing dam embankment is resting on alluvial deposits of sand, gravel, and clay from 4 to about 12 feet thick at the left abutment and valley floor, respectively. The right abutment is resting on bedrock. The alluvial deposits are underlain with a competent relatively impervious laminated sandstone. The core trench described earlier was designed to extend through these materials into the foundation bedrock.

Settlement

The construction plans indicate that the dam crest control was established at elevation 89.0 feet. An embankment profile was surveyed during the field investigation (Exhibit C2 of Appendix C). The survey shows that the maximum differential elevation along the main dam crest is about 0.7 feet. The point of greatest depression on the main embankment is located near the control tower. It appears the dam was designed without camber.

There is some slight misalignment along the outlet conduit, possibly the result of settlements. The settlements are small and are considered insignificant. In summary, the apparent settlement to date has not caused significant structural damage. Because the dam has aged nearly 27 years, additional significant amounts of settlement are not anticipated.

Seepage Control

Underdam seepage is controlled by a drain system installed at the base of the earth and gravel fill zone in the downstream portion of the dam. The plans indicate this drain was buried at a depth of about 6 feet and consists of an 8-inch pipe encased in gravel. The drain extends from the spillway chute to Station

3+40 (See Plan, Sheet 1 of 4, Exhibit C1). This system appears to be functioning properly as no seepage was observed coming from the embankment above the drain elevation. The drain outlet is buried in the stream channel and, therefore, the amount of flow could not be estimated.

At the time of the June 1980 field investigation, Cottonwood Reservoir was at the normal pool level (elevation 82.50 feet). Three general seepage areas were identified during this investigation; at the left abutment just below the downstream toe, downstream from the main embankment, and near the spillway stilling basin. The small seepage area on the left abutment (Photo 7 of Appendix B) is identified by saturated soil and lush water-type grasses. Free water was not evident at the time of this investigation. This area, as identified on Exhibit C3, appears to be located just beyond the end of the drain pipe.

The second seepage area is located about 250 feet downstream from the embankment at the base of the existing natural slopes (Photo 9 of Appendix B). The seepage in this area may be passing through the weathered zones in the laminated sandstone bedrock of the left abutment.

There is a spring or seepage area located immediately downstream of the spillway in the stilling basin area. This spring is identified in the files (Ref. 7) as existing prior to the construction of Cottonwood Reservoir. The spring is not discharging sediments or soil particles and appears to be insignificant to the safety of the dam.

In addition to these areas, seepage was identified in the area of the spillway chute. This area will be discussed in greater detail in connection with the discussion on the spillway (See Section 3.1.2, page 22). This particular seepage is related to the concrete spillway structure and not to under-the-dam or through-the-dam embankment seepage. This conclusion is based on two observatons made in the field. First, the center concrete floor section has raised relative to the outside edges of the chute floor. This movement has broken the water stops in the concrete floor. Water can enter under the spillway chute near the crest and run downslope under the concrete floor. Second, water was spilling over the spillway during part of this field investigation as a result of wind generated waves. During the investigation, the wind decreased and the flow in the spillway almost stopped. The seepage appeared to decrease proportionately with the decrease of water in the spillway.

2.3.3 Stability

Embankment

The slope angles measured during the field investigation are shown in Exhibit C3 of Appendix C. These angles were measured with an Abney level and should be considered approximate.

Except for some loss of riprap on the upstream slope and sloughing of the topsoil on the downstream slope, there is no outward sign of instability. However, there is no stability analysis data available to evaluate the slope design, and the phreatic surface in the embankment is unknown. Available design information suggests that soil strength tests were not performed and stability calculations were not made.

Small scarps are evident on the downstream face at several locations where topsoil has sloughed. The depth of the slope sloughing material is estimated to be less than about 1.5 feet. This condition is not considered to be of immediate danger. This topsoil sloughing on the downstream face may develop into a stability and/or erosion problem in time.

Erosion

Embankment and channel bank erosion were observed at several locations. Without corrective measures, these points of erosion may develop into stability problems.

At isolated locations on the upstream face of the embankment, wave action has eroded the embankment to low vertical cuts leaving the slope exposed and oversteepened (Photo 8 of Appendix B).

Accumulations of precipitation on the dam crest in the traffic path, combined with the vehicular traffic, has caused some ponding and created small erosion gullies (Photo 6 of Appendix B). A rather large depression has developed where the road (traffic) ascends the downstream slope of the dike.

Bank erosion was observed in the spillway stilling basin area (Photos 4 and 10 of Appendix B). The erosion is undercutting and exposing the end of the wing walls. This erosion does not seem to be affecting dam stability currently but could eventually affect the structural stability of the spillway chute if not corrected.

The reservoir shorelines are considered to be in stable condition as no major slides or scarpments were observed. The shoreline is

occasionally vertical, or near vertical, to heights of about 2 feet and localized sloughing occurs due to wave action and saturated conditions (Photo 1 of Appendix B).

2.3.4 Rock Riprap

Rock riprap has been placed on the upstream face of the dam, and in the stilling basin for the outlet works and spillway. This rock is sandstone. Some weathering is evidenced by the rounding of the edges and the decrease in size of the stone. Also the riprap has occasionally slipped down the upstream embankment face due to erosion and wear action. This riprap appears adequate presently, but should be monitored annually.

There is insufficient riprap protection in the stilling basin near the wing walls. The right (south) wing wall backfill has been badly eroded and undercut (Photo 10 of Appendix B).

There is some small gravel riprap on the upstream face of the dike. Although this material has slipped down the face of the dike at some locations, it appears adequate.

2.4 PROJECT OPERATIONS AND MAINTENANCE

The project is owned and administered by the Montana DNRC through a contractual agreement with the Shields Canal Company (Ref. 7). Initially, the State Water Conservation Board (SWCB) had the responsibility to construct, operate and maintain water conservation projects. In 1967, the SWCB was replaced by the Montana Water Resources Board (MWRB). The new agency shifted its emphasis from construction activities to providing engineering services for local groups and water user associations, to assisting in the maintenance of previously constructed projects, and to conducting annual maintenance inspections. Responsibility for project operation and maintenance gradually shifted from the State agencies to the local groups over a period of years. Upon reorganization of state government in 1971, the MWRB was replaced by the Department of Natural Resources and Conservation (DNRC). Accounting and engineering responsibilities were departmentalized, with the Engineering Bureau of the Water Resources Division, DNRC, delegated the responsibility to provide the services previously mentioned for the MWRB. At the present time, the Shields Canal Company performs essentially all operation and maintenance activities and the State agency serves only in a supervisory capacity (Ref. 7).

Representatives of the DNRC perform regular inspections of the project. They advise the Company of maintenance, repair, operation and budgetary needs. The DNRC provides assistance and direction in determining the best repair alternatives and financing sources (Ref. 7).

There is no formal reservoir operation plan, and essentially no records are kept on inflows, storage, and outflows. On May 23, 1967, the SWCB requested the project operator to assign an individual to measure and record the amount of storage in Cottonwood Reservoir. Measurements were to be made on the first and fifteenth of each month of every year to establish proof of water usage (Ref. 7). Evidently this assignment was never implemented as no records were available in the project file and the project operator does not currently measure pool levels.

Releases from the reservoir are generally dictated by water availability and seasonal water requirements. Seasons of particular interest are the spring and early-summer runoff period, and the irrigation period. Generally the project operator lowers the reservoir pool in the winter and early spring in anticipation of the spring runoff. The operator then traps all the spring, early-summer runoff possible for later release during the irrigation season. Releases during the irrigation season are made according to downstream water needs.

There are no formal maintenance and regular, mandatory inspection plans for the project. Even though some components of the project have required maintenance in the past, the work generally has not been accomplished due to manpower and budgetary constraints. Also, regularly scheduled inspections by the operator are not part of the operation. However, representatives from the Shields Canal Company generally attend project inspections which are scheduled by the DNRC. Water users are frequently in the vicinity of the storage project and maintain a general feeling for its condition, but do not regularly make a point to examine in detail the dam and appurtenances.

Only two (2) project modifications are known to have occurred on the Cottonwood Dam: one relates to a drilling program and the other relates to flashboards on the spillway. About one year after project completion, at least 9 grout holes were drilled. Nine drill logs were found in the project file (Ref. 7) and only two (2) were found in the field during the inspection. There is no record explaining the purpose of the drilling and whether a grout material was ever injected. It appears that a majority of the holes were placed along the dam crest.

The other modification occurred approximately 3 years after the original construction when timber flashboards were installed in the spillway to allow greater storage capability. Approximately 4 years ago the braces for the flashboard feature were replaced due to rotting.

There is no formal warning system or plan of action in the event of dam distress. Though not part of a formal plan, a distress condition would be communicated by phone, radios, and door-to-door contact.

CHAPTER 3
FINDINGS AND RECOMMENDATIONS

3.1 FINDINGS

Visual inspection of the dam, supplemented by analysis of the project in accordance with the guidelines (Ref. 1) and the contract performance standards, resulted in the following findings.

3.1.1 Size, Hazard Classification and Safety Evaluation

In accordance with the inspection guidelines (Ref. 1), Cottonwood Dam is classified intermediate in size and, based on our visual inspection and engineering judgment, it has a high downstream hazard potential. Therefore, the guidelines' recommended spillway design flood (SDF) for this project is 100 percent of the PMF. Based on reconnaissance level investigations, the project is incapable of handling a flood having one-half the PMF ordinates because the project can safely handle only a flood having 12 percent of PMF hydrograph ordinates without overtopping and causing the dam to fail which, in our judgment, would seriously jeopardize life and property downstream. Under inspection guideline criteria, Cottonwood Dam has a seriously inadequate spillway and is considered unsafe-nonemergency until the recommended actions are completed.

3.1.2 Spillway

The spillway system was designed to accommodate a relatively small flood event compared to current standards. Maximum spillway capacity, assuming the reservoir pool is at the first overtopping dam crest elevation (elevation 88.74 feet), is approximately 1030 cfs. Spillway capacity could be approximately doubled by removing the flashboards. The floodwater storage capacity between the spillway crest with flashboards (elevation 82.5 feet) and the first overtopping dam crest elevation (elevation 88.74 feet) amounts to 1770 AF. In comparison, the PMF for the 34 square-mile drainage area is estimated to have a total 72-hour runoff volume of about 20,210 AF. Hence, the combination of reservoir storage and spillway discharge capabilities is inadequate to prevent overtopping of the dam during the PMF.

The spillway is in poor condition. The catwalk across the spillway crest is in need of repair. The wooden flashboard supports could accumulate debris and block the spillway. The concrete chute has become very rough due to erosion and severely cracked in places as a result of settlement and/or uplift. Near the top of the chute, cracking and spalling along the

construction joints has exposed the metal water stops permitting water to seep underneath the spillway floor. Water is escaping under pressure from underneath the floor slab at the bottom of chute causing a displacement of approximately three inches in the floor slab. This water pressure is likely to cause failure of the floor slab if not corrected. Failure of the floor slab could endanger the dam embankment. Settlement, uplift, and cracking has also occurred in the stilling basin walls and baffle blocks. Erosion is taking place at the stilling basin wing walls, most severely on the right (south) side, and silt and riprap have deposited in the stilling basin floor. The spillway is probably not capable of passing its estimated maximum discharge of 1030 cfs without failure of the floor slab.

3.1.3 Outlet Works

The outlet works facility appeared to be in satisfactory operating condition from the control tower to the outlet. The inlet structure and 36-inch conduit from the inlet to the control tower could not be inspected. Repair is required in the following areas: on the operating gate to allow proper seating, at the joint between the CSP and the gate, at the contact between the CSP and the cast iron vent pipe, on the interior asphaltic pipe coating, on the paved invert of the pipe, at the pipe joints where the joint-filler material is removed, and on the concrete where the CSP daylights at the stilling basin.

3.1.4 Dam

Except for the loss of some riprap in the stilling basin and on the upstream face, the Cottonwood Reservoir embankment appears to be stable and in good condition. Some questions exist as to the embankment stability because insufficient information is presently available to fully evaluate the dam. Settlement does not appear to be a problem. Three seepage areas were identified, but all were considered relatively insignificant. The seepage appears to be passing through the abutments or under the dam. The phreatic surface through the dam is unknown. The upstream face of the embankment and the outlet channel sideslope in the stilling basin near the wing walls are deficient in riprap protection. In some of these areas, the riprap has moved down the slope exposing erodible soils. The dam crest is rounded on the upstream side and there are some depressions along the crest which collect water and provide a concentrated source of water for spillage over the dam face. Debris on the upstream face of the dam was limited to very small material.

3.1.5 Operation and Maintenance

There are no formal operation plans, maintenance programs, and/or mandatory inspection programs. The storage project is visited regularly throughout most of the year by the operator to regulate water releases. Project inspections are occasionally performed by the DNRC, and generally representatives of the Shields Canal Company attend. Minimal repair work has been accomplished in the past. In fact, most project components have not been maintained or repaired when first required. The basic reason for the meager maintenance program and repair efforts is the ever-present manpower and budgetary limitation. A warning system to alert downstream inhabitants of dam distress has not been developed.

3.2 RECOMMENDATIONS

(1) Immediately develop, implement, and periodically test an emergency warning plan for use in the event of dam distress.

(2) Perform the following repairs on the spillway as a temporary measure until more advanced engineering studies and field studies are performed (Recommendations #9 and #10): remove the flashboards to approximately double the spillway capacity; repair the concrete floor in areas where the slabs have shifted, holes have developed, and water stops have become exposed in order to keep excess water from flowing beneath the spillway floor; and replace the catwalk with a metal structure having safety rails, or at least, replace the hand rails on the existing wooden structure.

(3) Practice maximum reservoir management to prevent large discharges from flowing through the existing spillway until it is repaired or replaced.

(4) Perform the following repairs on the spillway stilling basin: remove the rock from between the baffle blocks; rearrange/replace the rock riprap downstream of the blocks and near the basin wing walls; and repair the concrete cracking of the blocks and walls.

(5) Inspect the outlet works from the valve upstream to the intake structure, and perform the necessary repairs.

(6) Perform the following repairs on the outlet works downstream of the valve: repair the gap in the 36-inch CSP pipe immediately downstream of the valve; repair the gate seat; clean and recoat the metal pipe interior with a protective coating; replace deteriorated portions of the paved invert

along the flowline of the pipe; replace the joint filler material at the pipe joints; apply a protective material to the contact area between the cast iron vent pipe and the 36-inch CSP to stop localized corrosion; and apply a filler or grout material along the stilling basin wall where the outlet works pipe daylights.

(7) Replace/rearrange rock riprap along the upstream face of the embankment where the rock has moved downslope.

(8) Using suitable material, fill and properly compact all depressions along the dam crest.

The above recommendations will not make the project safe but will reduce involved risks while the following recommendations with subsequent actions are being accomplished.

(9) Conduct more detailed hydrologic and hydraulic routing studies to better determine the downstream hazard and required spillway capacity and modify the project as studies indicate.

(10) Perform field testing on the spillway to determine the foundation condition beneath the spillway and stilling basin floor. These tests should be assessed before any major repair is performed on the spillway.

(11) Make an assessment of the phreatic surface through the dam by drilling and installing a few piezometers. Perform and place on file a slope stability analysis of the embankment. This analysis should be conducted by a qualified geotechnical engineer and be based on actual phreatic surface and strength characteristics as determined from piezometer drilling and sampling. Modify the dam sections as required for stability according to guideline criteria.

(12) Conduct periodic inspections by qualified engineers at least once every five years to determine whether there are any deficiencies in the condition of the project, to assess the adequacy and quality of maintenance, and to evaluate methods of operation. Include an inspection of the total length of the conduit through the embankment in this program.

(13) Develop and implement a periodic maintenance plan for the dam and appurtenant structures.

The project operator should contact the Montana, DNRC, Dam Safety Section, prior to performing any remedial construction to insure compliance with all pertinent laws and regulations.

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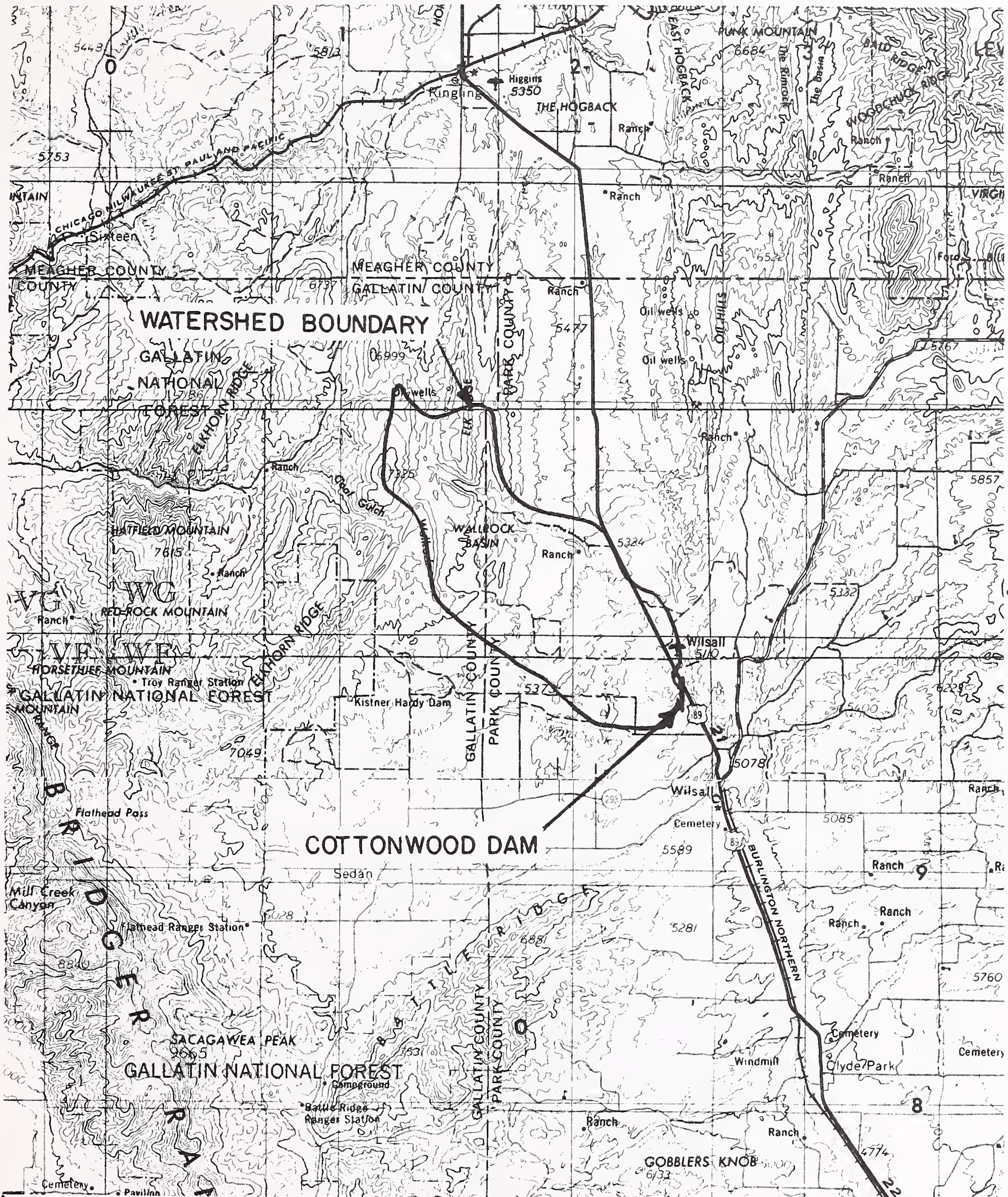
APPENDIX A

VICINITY & WATERSHED MAP



4 0 4 MILES

SCALE

SOURCE: WHITE SULPHUR SPRINGS &
BOZEMAN, MONTANA AMS MAP
U.S.G.S.

N

APPENDIX A

VICINITY & WATERSHED MAP

COTTONWOOD DAM

APPENDIX B

INSPECTION PHOTOS

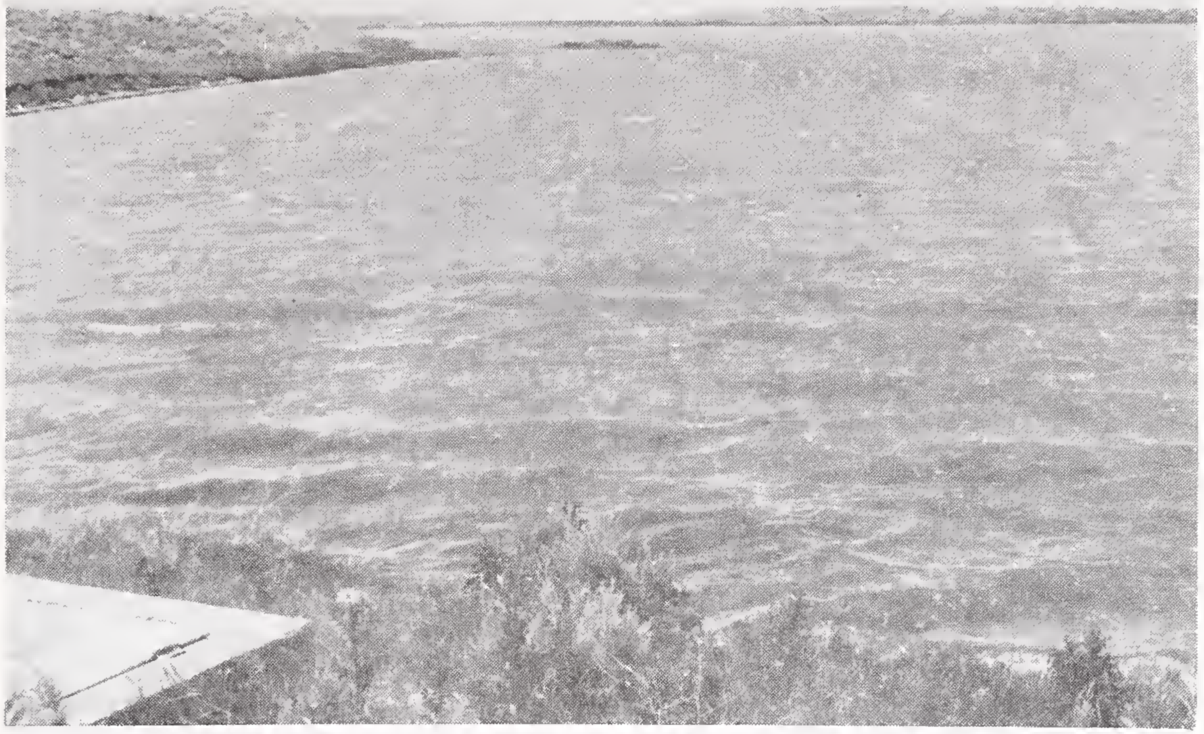


Photo No. 1 - Upstream Watershed
The reservoir banks are sloughing at the upper left side of the photo.

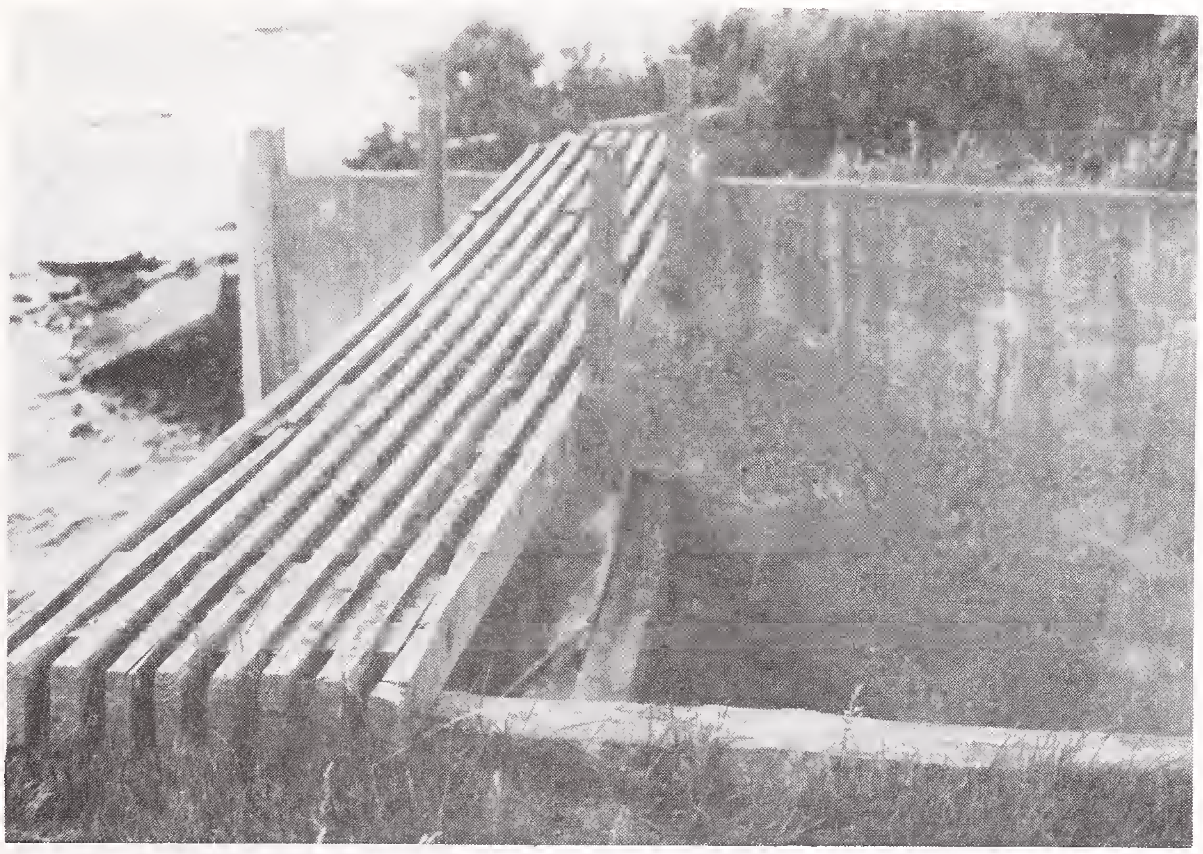


Photo No. 2 - Catwalk over Spillway
The catwalk is in poor condition and missing handrails.

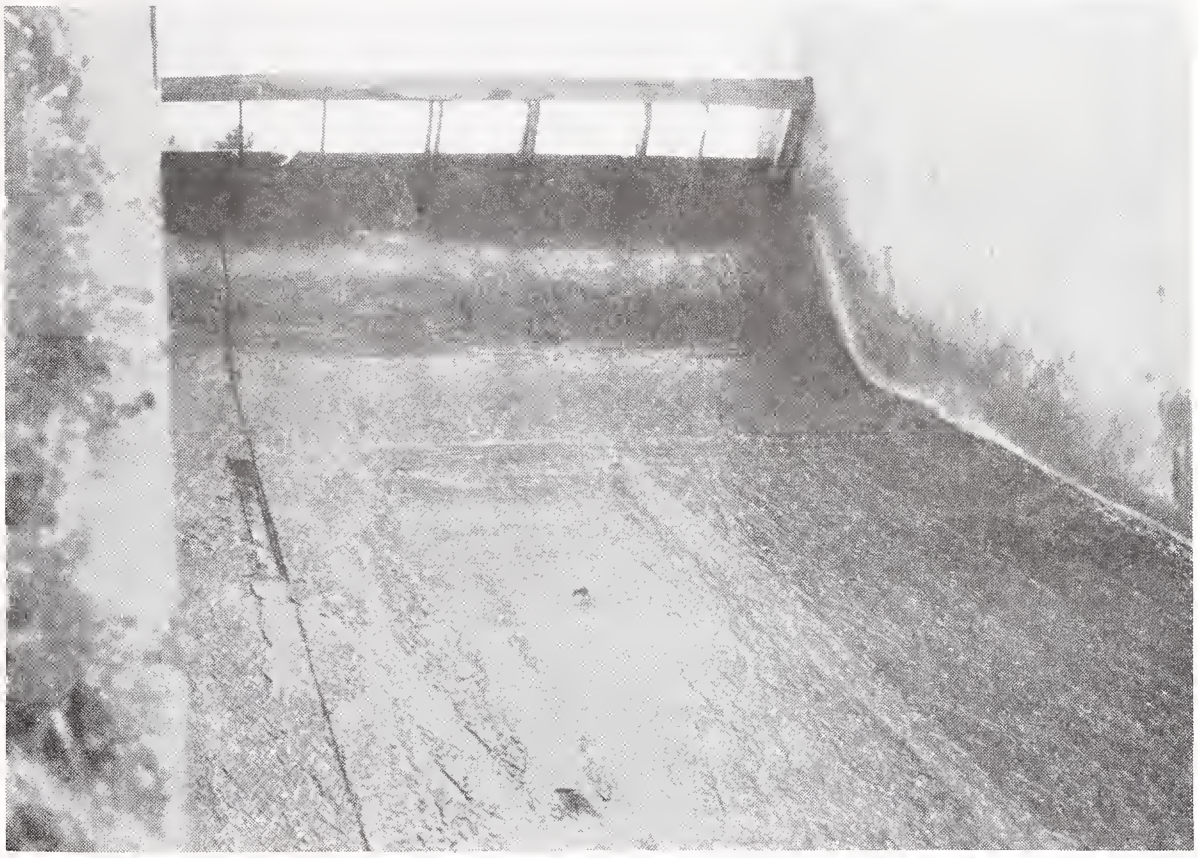


Photo No. 3 - Top of Spillway
The spillway floor is cracked exposing metal water stops.

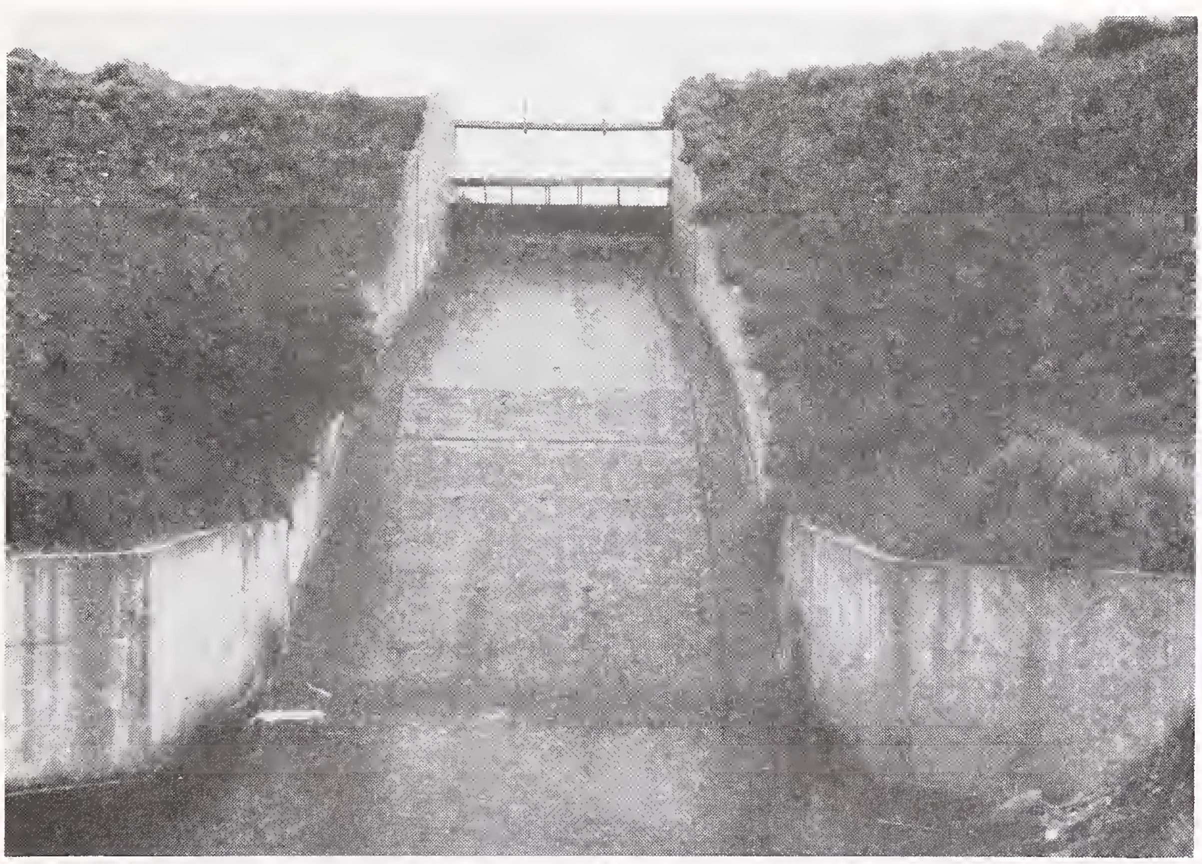


Photo No. 4 - Spillway
Water under pressure is escaping from beneath the bottom of the spillway floor slab.

Photo No. 5 - Outlet Pipe
A large crack in the stilling basin wall exists just upstream of the outlet pipe exit. There is a gap between the outlet pipe and the concrete in the stilling basin wall.

Photo No. 6 - Dam Crest
The dam crest is shown looking from north to south.

Photo No. 7 - Downstream Dam Embankment
The downstream side of the embankment is shown looking towards the south.

Photo No. 8 - Upstream Dam Embankment
Shown is the upstream face of the embankment looking northerly.

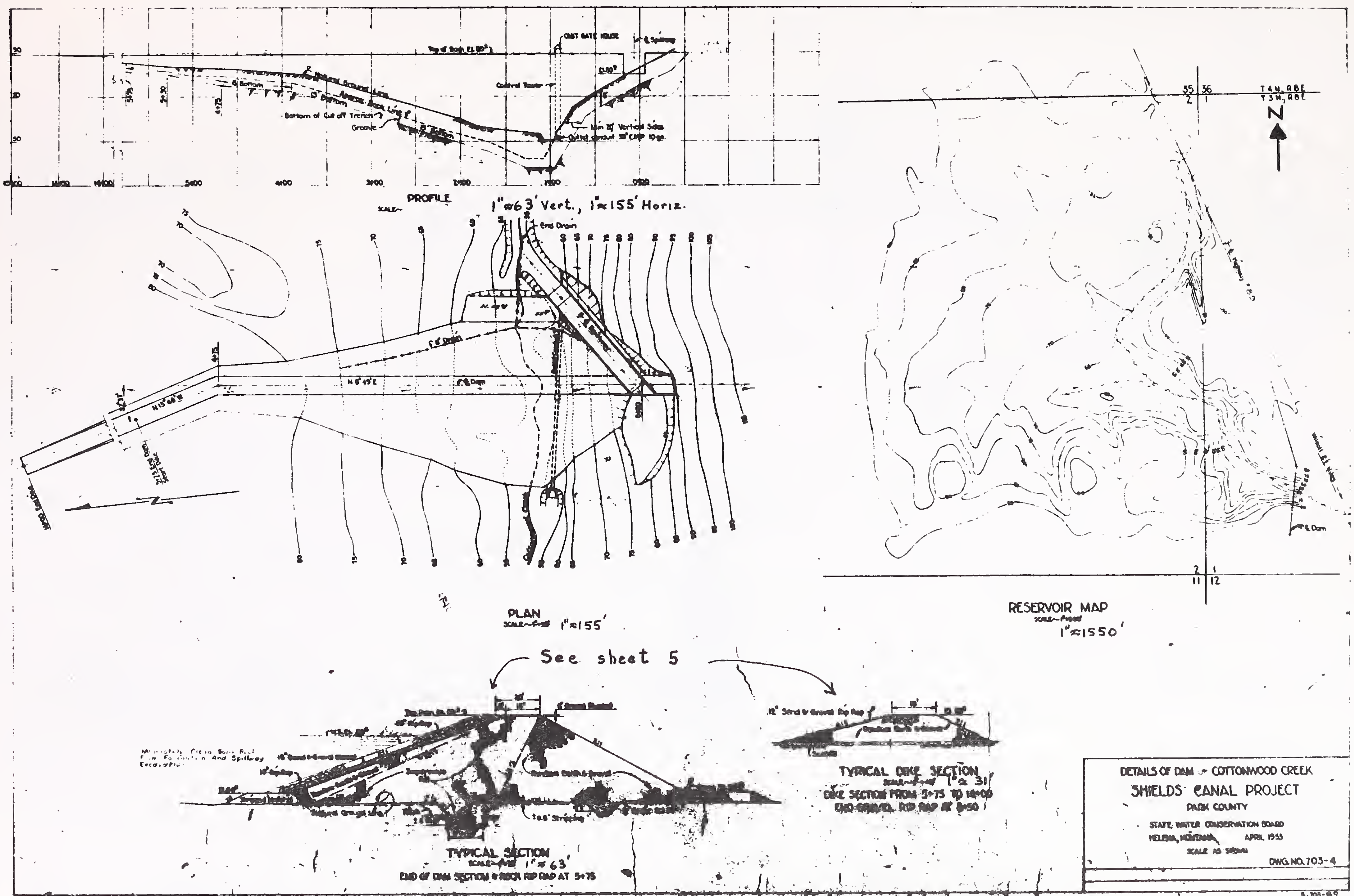
Photo No. 9 - Seepage Downstream of Dam
Shown is the seepage through the left abutment approximately 250 feet downstream of the dam toe.

Photo No. 10 - Spillway and Outlet Works Return Channel
Erosion is taking place at the stilling basin wingwalls. The highway embankment is 400 feet downstream of the dam embankment.

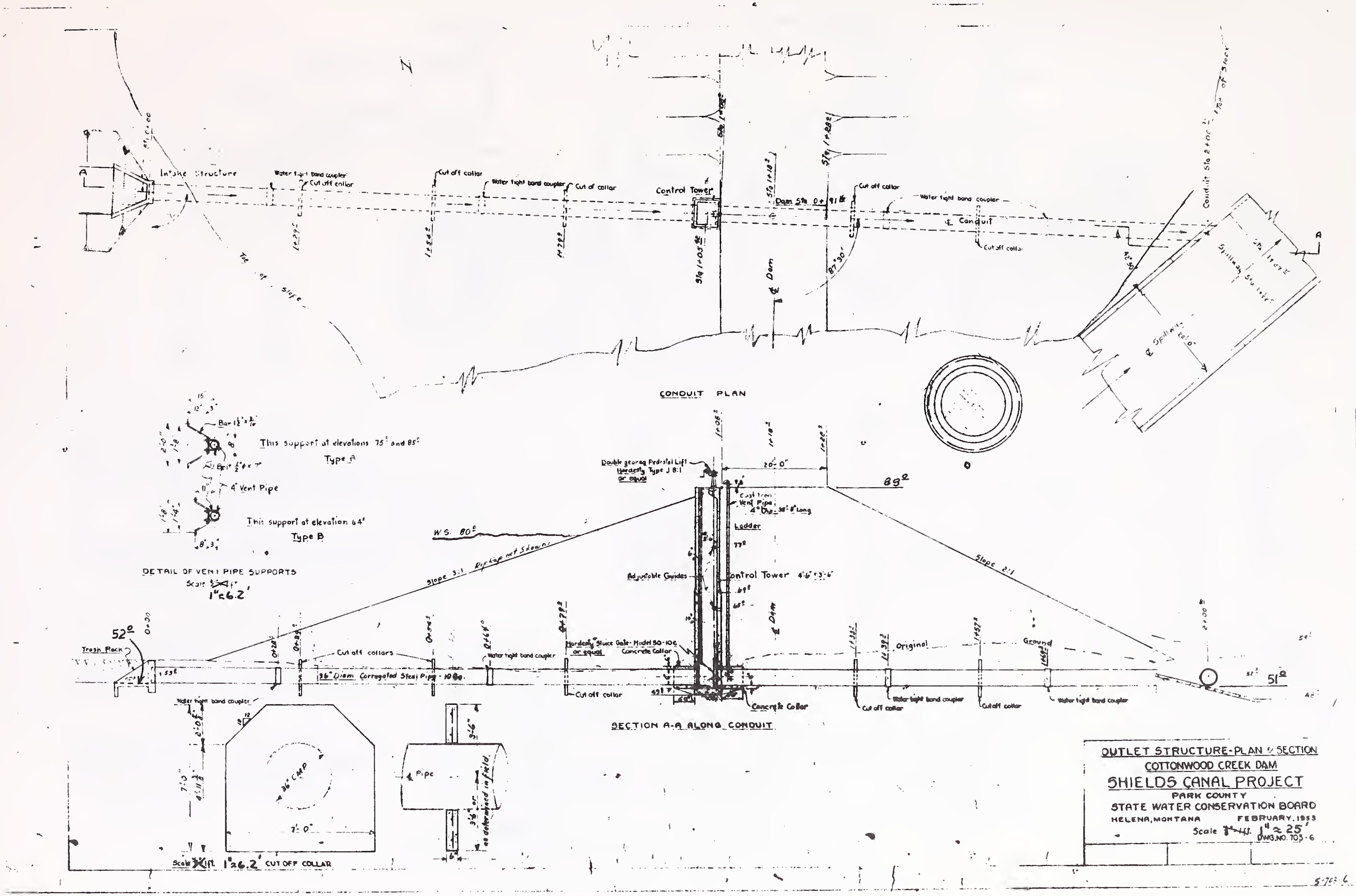
APPENDIX C

PROJECT DRAWINGS

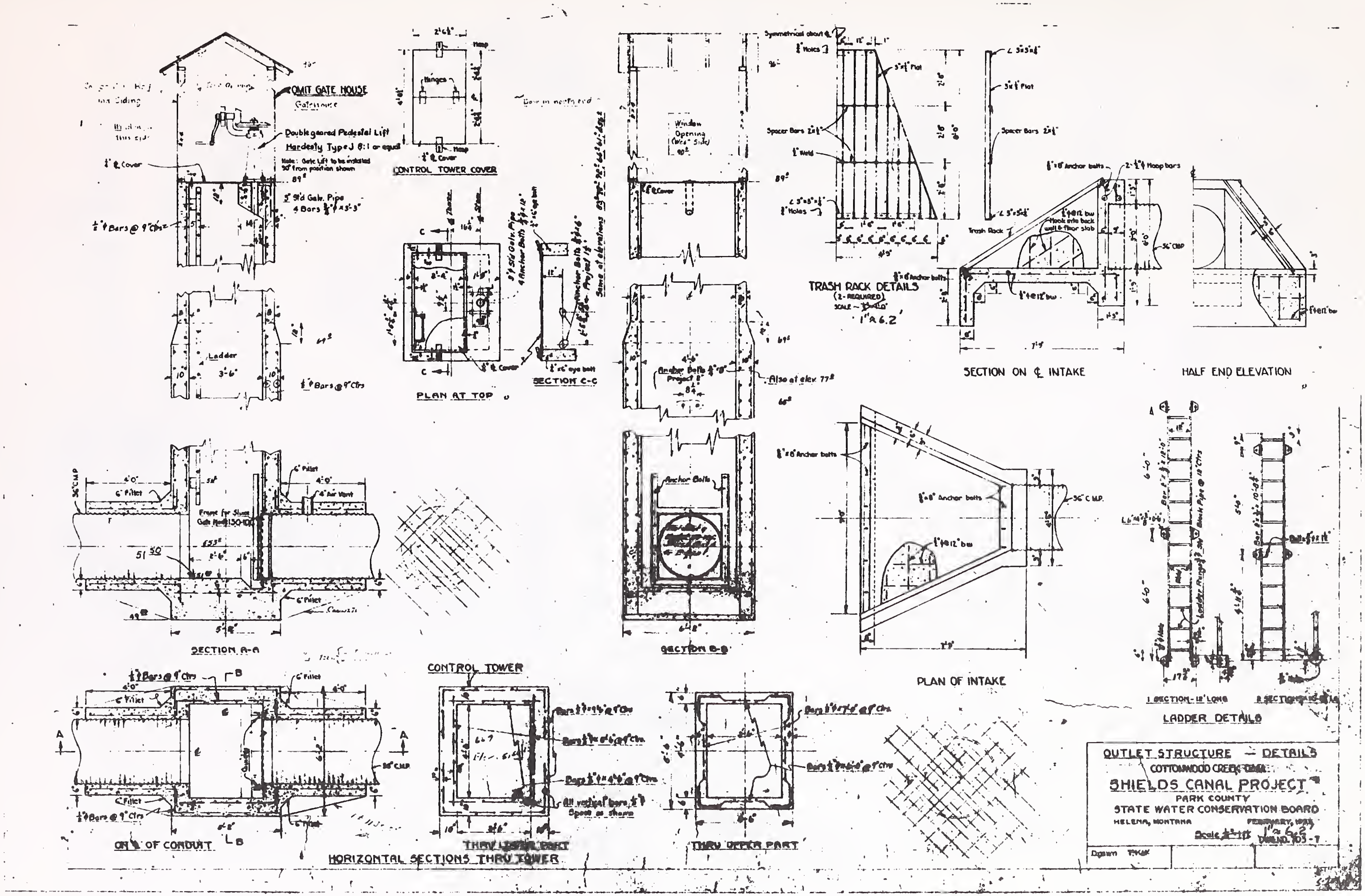
Exhibit C1	Construction Plans
Exhibit C2	Dam Crest Profile
Exhibit C3	Measured Embankment Slopes



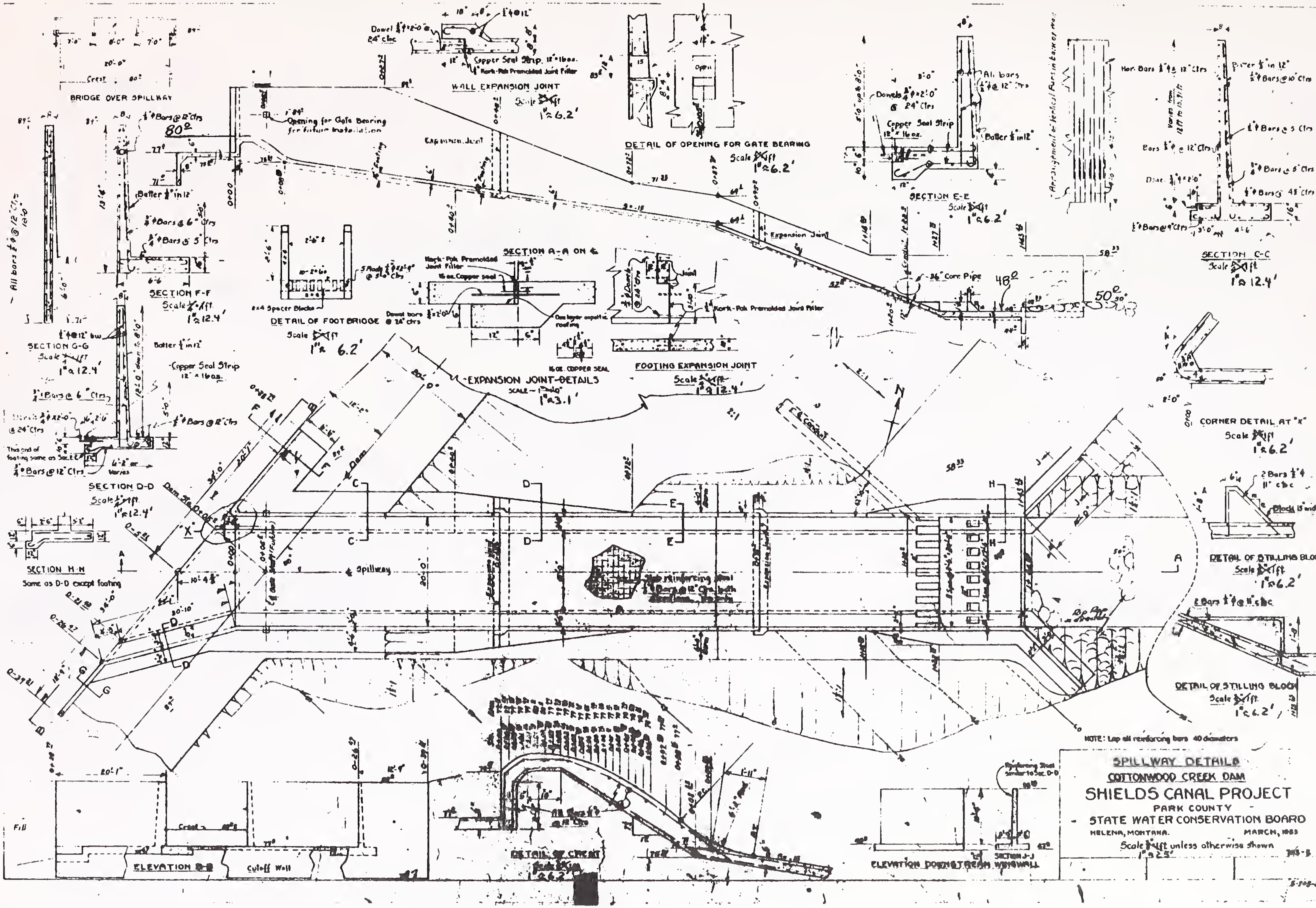
SHEET 1
EXHIBIT C1



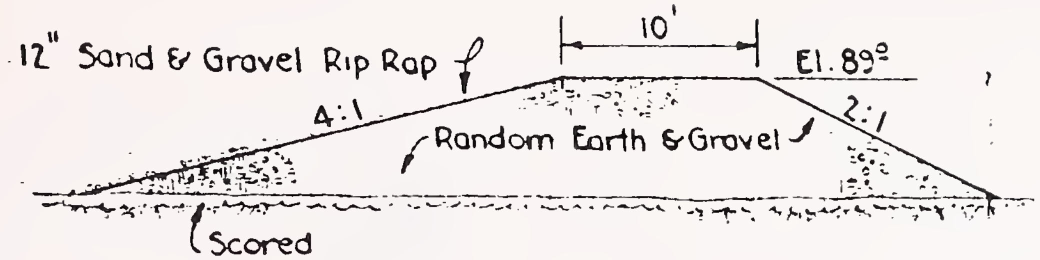
SHEET 2
EXHIBIT C1



SHEET 3
EXHIBIT C1



SHEET 4
EXHIBIT C1

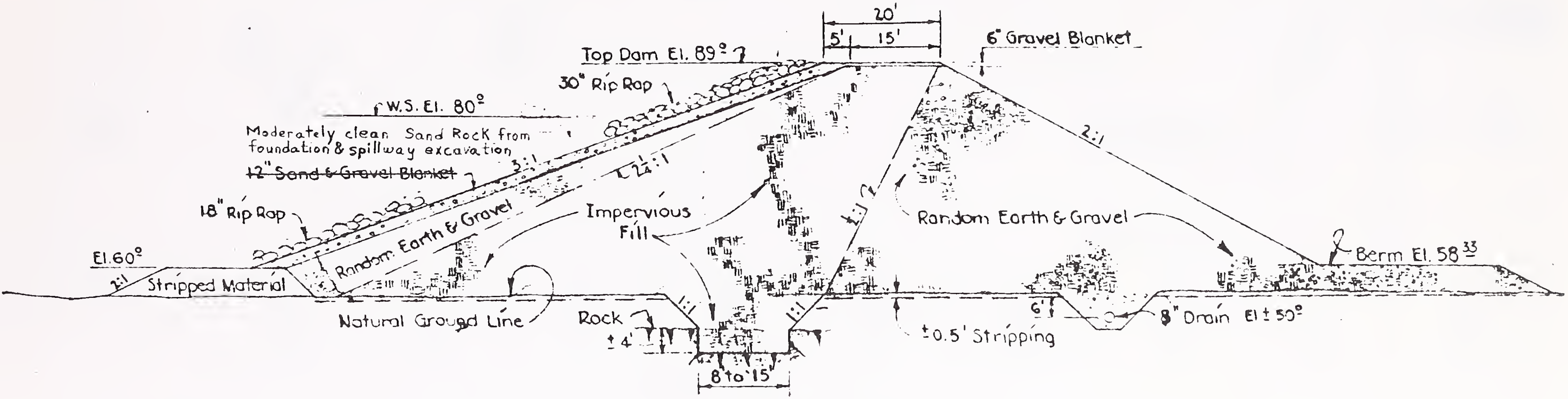


TYPICAL DIKE SECTION

SCALE ~ 1" = ~~10~~ 12.5'

DIKE SECTION FROM 5+75 TO 14+00

END GRAVEL RIP RAP AT 8+50



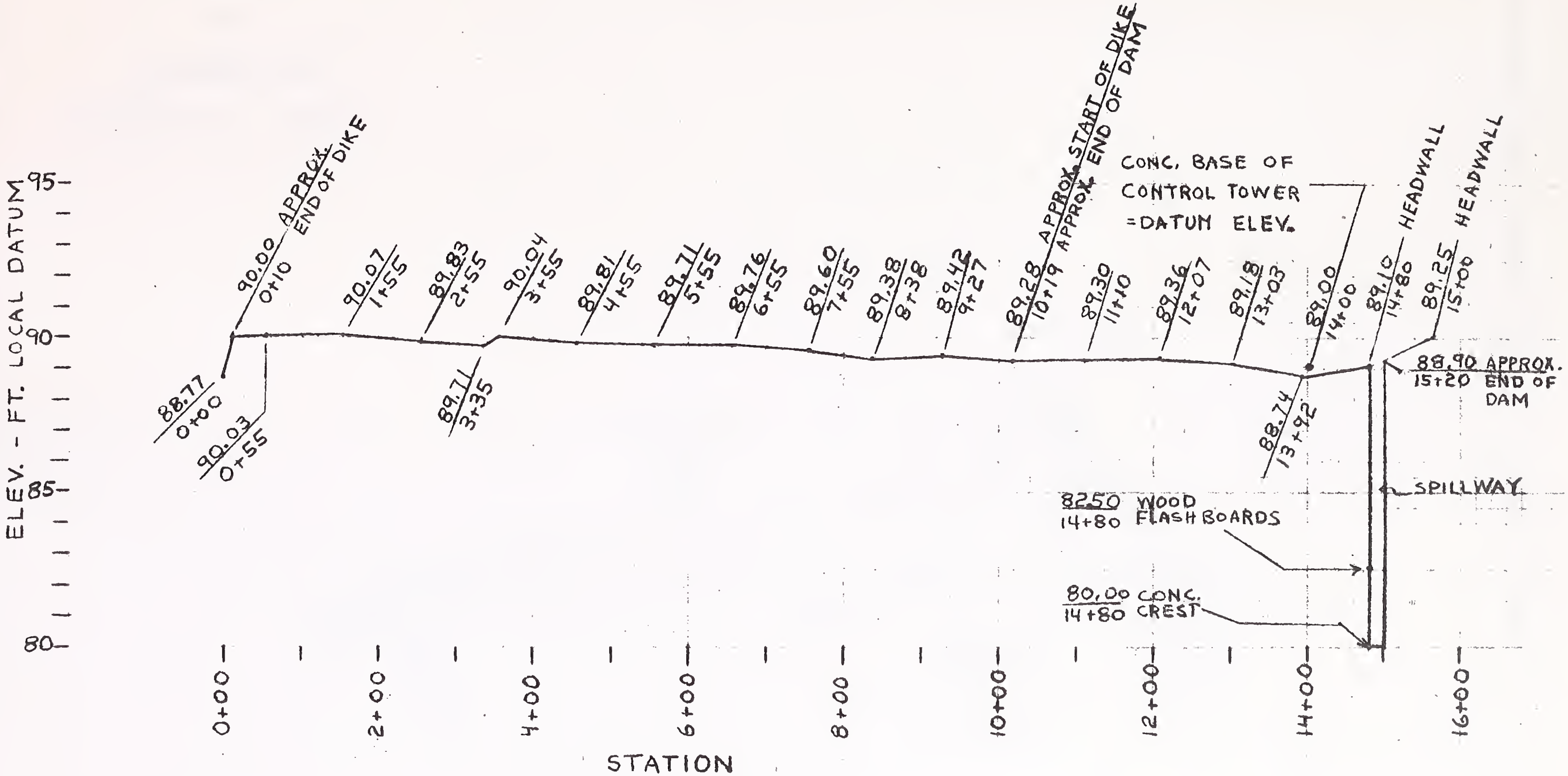
TYPICAL SECTION

SCALE ~ 1" = ~~20~~ 25'

END OF DAM SECTION & ROCK RIP RAP AT 5+75

SHEET 5

EXHIBIT C 1



NOTES

1. ELEVATIONS BASED ON CONC. BASE OF CONTROL TOWER = ELEV. 89.00 LOCAL DATUM
2. STATIONS SHOWN ARE FROM HKM ASSOC. SURVEY & DO NOT MATCH STATIONS SHOWN ON THE CONSTRUCTION DRWGS.
3. DATE OF SURVEY : 6/27/80

EXHIBIT C2
DAM CREST PROFILE
COTTONWOOD DAM

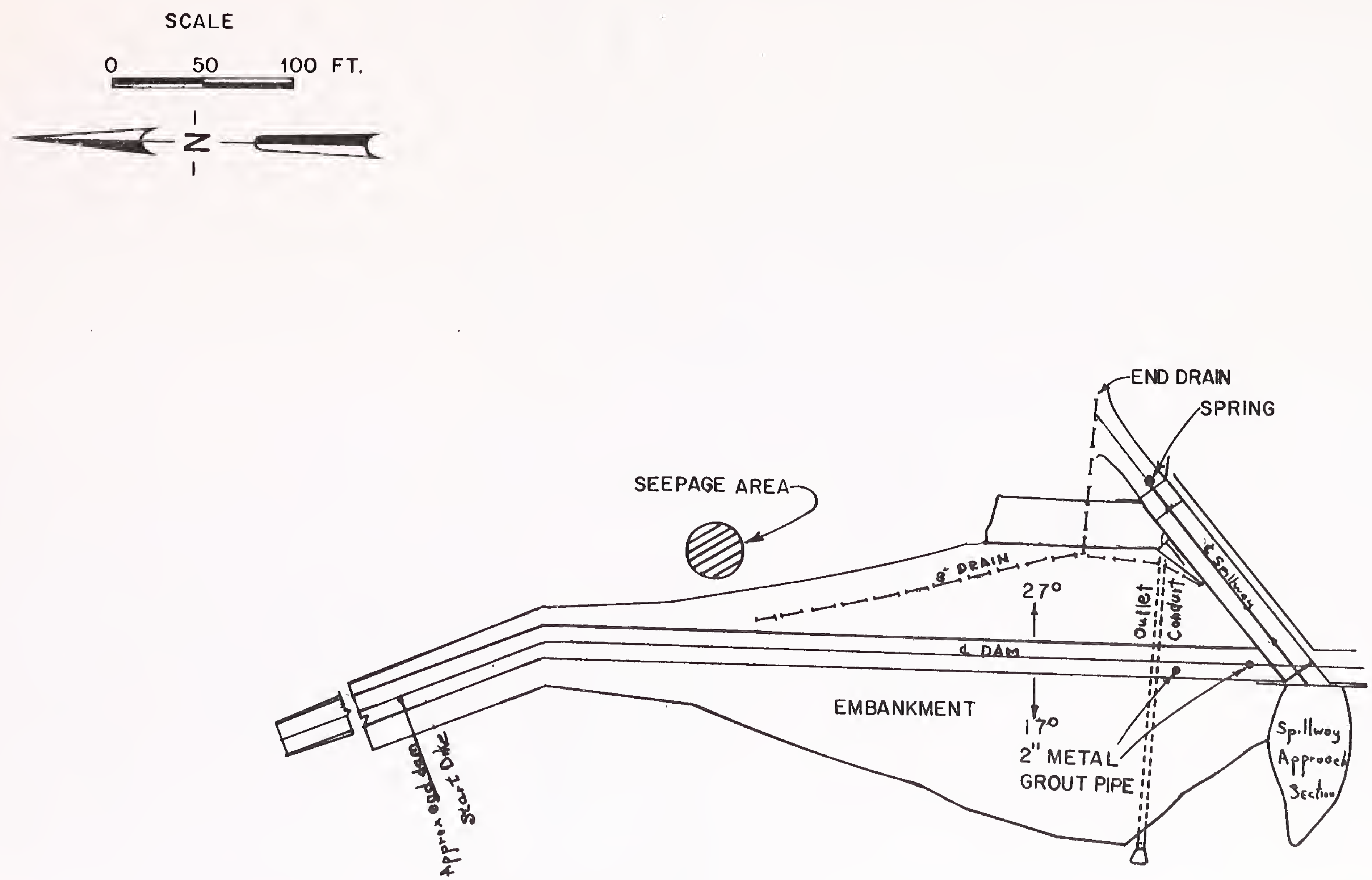


EXHIBIT C3

MEASURED EMBANKMENT SLOPES

COTTONWOOD DAM

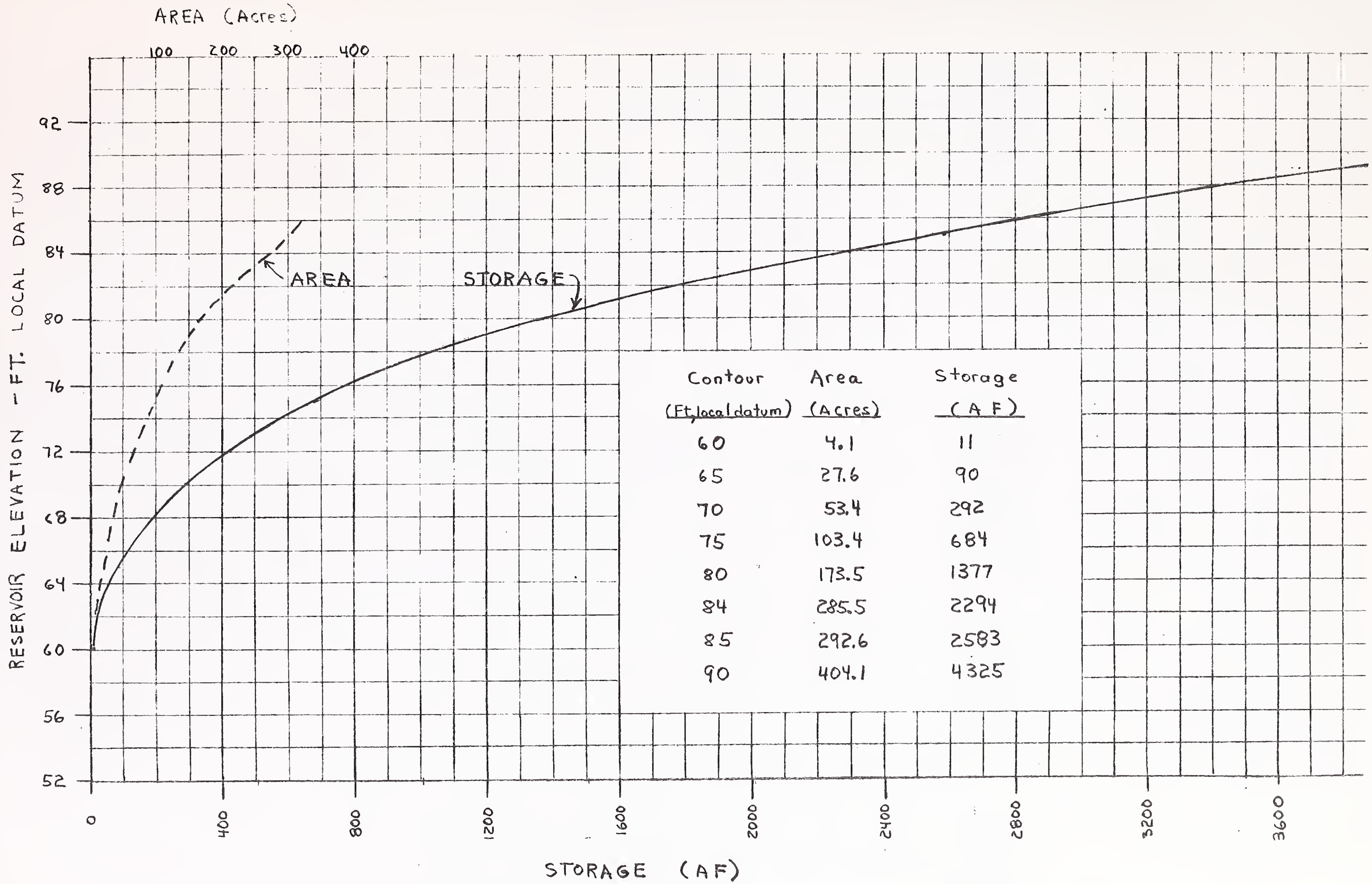
HKM ASSOC.

7M140.109

APPENDIX D

ENGINEERING DATA

Exhibit D1	Elevation-Area-Storage Curves
Exhibit D2	Discharge Rating Table
Exhibit D3	Discharge Rating Curves



Contour (Ft, local datum)	Area (Acres)	Storage (A F)
60	4.1	11
65	27.6	90
70	53.4	292
75	103.4	684
80	173.5	1377
84	285.5	2294
85	292.6	2583
90	404.1	4325

SOURCE: STATE WATER CONSERVATION PROJECTS, MARCH, 1977
MONTANA DNRC, WATER RESOURCES DIVISION

EXHIBIT D1
ELEVATION-AREA-STORAGE CURVES
COTTONWOOD DAM

EXHIBIT D2

DISCHARGE RATING TABLE

COTTONWOOD DAM

RESERVOIR ELEVATION (FT.LOCAL DATUM)	OUTLET WORKS DISCHARGE (CFS)	SPILLWAY DISCHARGE (CFS)	TOTAL DISCHARGE (CFS)
52	0	-	0
60	55	-	55
70	81	-	81
82.5 (SPILLWAY CREST)	105	0	105
83	106	25	131
84	107	120	227
85	108	260	368
86	109	430	539
87	110	630	740
88	111	850	961
88.74 (LOW PT. DAM CREST)	112	1030	1142
89	112	1095	1207

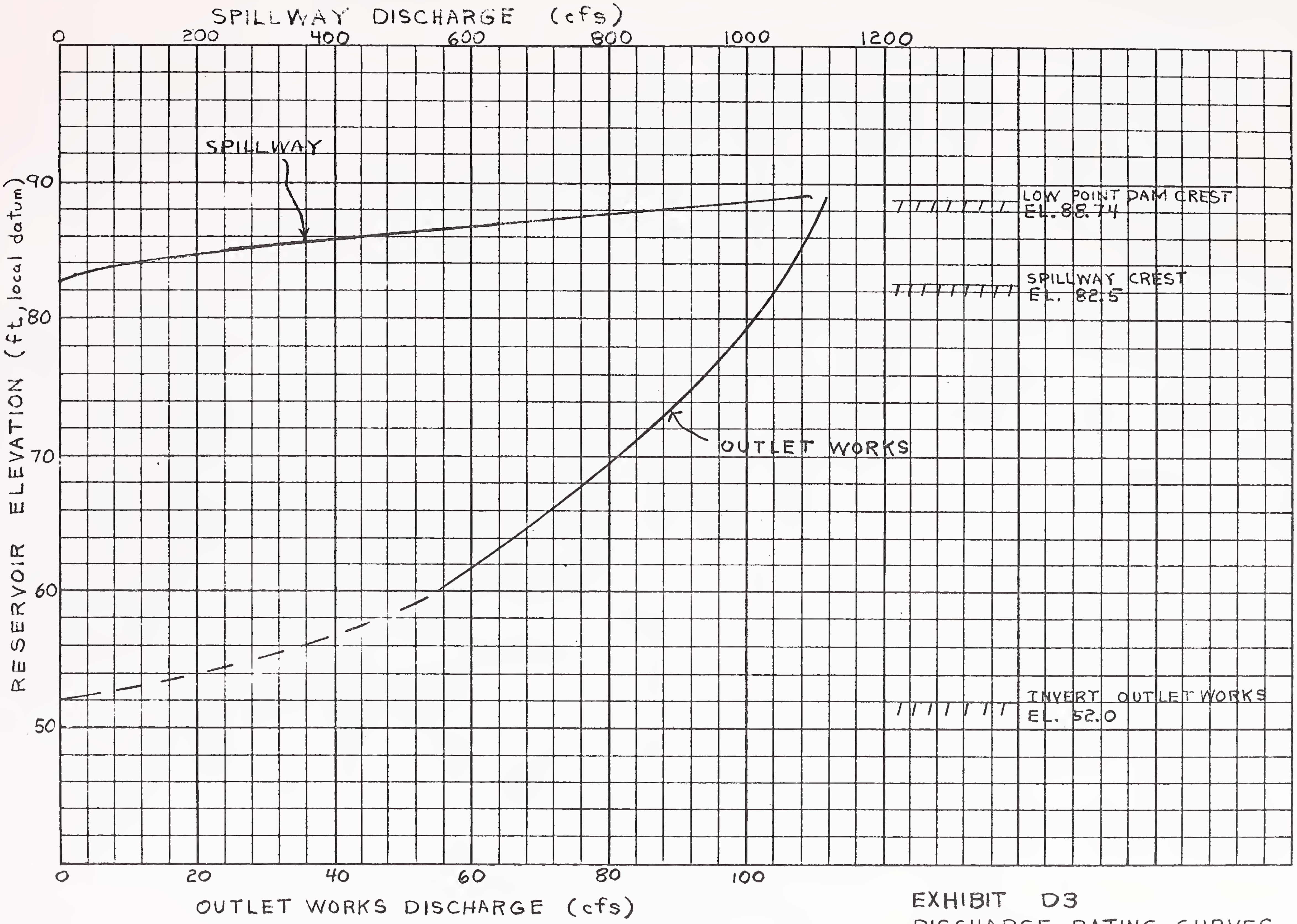


EXHIBIT D3
DISCHARGE RATING CURVES
COTTONWOOD DAM

APPENDIX E

CORRESPONDENCE

DEPARTMENT OF NATURAL RESOURCES
AND CONSERVATION
WATER RESOURCES DIVISION

THOMAS L. JUDGE, GOVERNOR

32 SOUTH EWING

STATE OF MONTANA

(406) 449-2872

HELENA, MONTANA 59601

January 22, 1981

Ralph Morrison
Department of the Army
Seattle District, Corps of Engineers
P.O. Box C-3755
Seattle, Washington 98124

Dear Mr. Morrison:

The Department of Natural Resources and Conservation has reviewed the final draft report on the Cottonwood Reservoir Dam (MT-21). We concur with the findings and recommendations in the report and feel that the report satisfies the criteria for the Phase I investigation. Our comments have been discussed with your staff and we understand that they will be incorporated into the final report.

We sent copies of the final draft report to the Shields Canal Company for their review and comments. Their comments are inclosed.

Thank you for the opportunity to review and comment on the final report for this project.

Sincerely,

Richard L. Bondy

Richard L. Bondy, P.E.
Chief, Engineering Bureau
(406) 449-2864

RB:LT:lj
enclosure

RECEIVED
DEC 30 1980
MONT. DEPT. OF NATURAL
RESOURCES & CONSERVATION

December 22, 1980
Rte. 85, Box 4250
Livingston, Montana
59047

Lawrence H. Tegg
Engineering Bureau
Dept. Nat. Resources & Conservation
Helena, Montana

Dear Sir:

In reply to your letter of Dec. 8 about the report of inspection, we do realize that there are several minor repairs needed to be done. The main reason we haven't done some repair is the fact it requires draining of the reservoir. That could leave us very short of water at a time when it is sorely needed. There have been years in the past that the dam hasn't filled.

As for removing the flashboards as stated in par. 2, p. 46, that would cut down the water supply too much. The dam was originally designed for a 36" high radial gate, so why are the present flashboards such a hazard?

Also we think the "downstream hazard" stated (after a brief inspection & judgement of the engineers - quote the report) is not that great and needs better grounds to decide as stated.

To our knowledge, there has never been

a run-off in that area that has caused any over-flow problem with this project in all the years since installation.

Sincerely,

Carl Arthun, Pres.
Shields Canal Company
Rt. 85, Box 4250
Livingston, Mont.
59047